J.M. Galvin

# Ground Engineering

## Principles and Practices for Underground Coal Mining

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J.M. Galvin

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J.M. Galvin
Manly, NSW, Australia

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# Foreword

Underground coal mining in Australia began at Newcastle, New South Wales, in the early 1800s. For almost a century, mining was by traditional hand working methods until the first mechanised coal cutter was introduced in 1890. More highly productive mechanised longwall mining was introduced in 1963 and is now the predominant method of Australian underground coal production. In common with underground mining in other parts of the world, during its 200 year history, underground coal mining in Australia has experienced a number of major disasters involving fatalities. However, particularly in recent decades, the Australian mining industry has a proud record of having reduced progressively the overall numbers and unit rates of fatalities and serious injuries arising from underground ground control issues.

Nevertheless, the issue of safety in underground coal mines remains of concern to the industry itself, to mining regulators, to those working in the industry in a range of capacities and to the community at large. Despite the advances that have been made in mine geotechnical engineering over the last 50 years through research and development and through advances in mining practice, it is widely accepted that the ground engineering and associated risk management aspects of underground coal mining still require the development of deeper basic understandings and the implementation of those understandings in mining practice. In response to these concerns, the industry developed the view, largely through its Australian Coal Research Association Program (ACARP), supported by the Minerals Council of Australia (MCA), that a handbook on ground engineering risk management in underground coal mining should be prepared.

The question of who should be commissioned to write such a handbook or textbook (as it became) gave the industry very little pause for thought. In terms of his depth and breadth of knowledge, his experience and his standing in the industry, Emeritus Professor Jim Galvin was the obvious choice.

After completing degrees in Science and Mining Engineering at the University of Sydney in 1973 and 1975, respectively, Jim Galvin worked in the South African mining industry where he obtained his PhD in mining rock mechanics from the University of the Witwatersrand in 1981. He went on to serve as Head of the Coal Strata Control Section of the South African Chamber of Mines Research Organisation. In 1982 he returned to Australia where he gained practical experience in all aspects of underground coal mining from face worker to mine manager. During this time, he served as a Member of the NSW Mines Rescue Service. Jim was appointed Professor of Mining Engineering at the University of New South Wales in 1993 and served as Head of School from 1995 to 2003. He now has an extensive private consulting practice. Jim was elected a Fellow of the Australian Academy of Technological Sciences and Engineering in 2009.

Throughout his career, Jim has had a special interest in risk management, particularly as it applies to workplace health and safety and the environment. He has served in a range of expert capacities at state, national and international levels in mine accident investigations, in planning enquiries, in the provision of expert evidence, in review roles for mining companies and regulators and in delivering courses and keynote lectures. These roles include serving as Chair of the Victorian Government's mining Technical Review Board; an Independent Advisor to the Health, Safety and Environment Committee of the Board of BHP Billiton; Safety Advisor to the Board of Solid Energy, New Zealand; a Statutory Member of the NSW Planning Assessment Committee; an International Expert Reviewer for the Mine Health and Safety Council of South Africa; and Chair of the Continuing Professional Development Committee of the Mine Managers Association of Australia.

During the course of the preparation of this book, I had the opportunity to review every chapter and to discuss with Jim several of the important questions that his text addresses. In my opinion, the book provides an outstanding, detailed and much needed, account of ground engineering principles and their application in underground coal mining practice in Australia and internationally. A particular strength of the book is the way in which good underground coal mining practice is identified and discussed within an understandable and logical applied mechanics framework. It provides a fine example of what good mining engineering should be. As my fellow reviewer, Emeritus Professor Horst Wagner, has said, "a particular and unique aspect of the book is the link between ground engineering and risk management......there is no comparable text which covers ground engineering principles and underground coal mining practice in such a comprehensive way".

I congratulate Emeritus Professor Jim Galvin for an outstanding achievement. I recommend this book unreservedly to all those having responsibility for identifying and managing ground control-related risk issues in underground coal mines, including mine managers, planners, operators, geotechnical engineers (including consultants), mining regulators, academics and especially mining engineering students. It is my hope that the rational approaches discussed in this book will replace the largely empirical methods used for coal mine excavation design in Australia and internationally.

Golder Associates Pty Ltd., Brisbane, Australia
University of Queensland, Brisbane, Australia
President of the International Society for
Rock Mechanics, 1983–1987
9 March 2015

Edwin T. Brown AC

# Preface

Ground engineering is a critical component in designing and conducting mining operations that are safe, efficient and economically viable. Its contribution is characterised by pervasive uncertainty due to an incomplete knowledge of material properties, behaviour mechanisms, loading environments and the strength of rock structures. Consistent with international standards, the effect of this uncertainty on achieving objectives constitutes risk. This means that ground engineering should be practised within a risk management framework that aims to both prevent unwanted outcomes and mitigate their consequences to an acceptable level. To be successful, this process requires knowledge of fundamental scientific and engineering principles relevant to ground behaviour; knowledge of mining systems, practices and hazards; and an understanding of risk management principles, supported by experience and skill.

This text has its origins in a request from the Australian coal mining industry to develop a ground control risk management handbook from the perspective of both an academic and a mine operator and, in the process, to clarify a range of conflicting and confusing advice to the industry regarding ground control practices. It soon became apparent that in order to achieve this goal in a manner that was objective and consistent with risk management processes, there was a need to re-establish the basic principles of rock behaviour and to apply these to practical mining situations. This task evolved into one of writing a textbook that aims to provide ground engineering principles and practices associated with underground coal mining at a technical level and in a language and format appropriate to ground control practitioners and to those that engage with these practitioners.

The text is written by a mining engineer with a specialist knowledge in rock mechanics and risk management and who has had practical experience, responsibility and accountability for the design and management of large underground coal mines and for the consequences of loss of ground control. Hence, its audience is wide ranging and includes geoscience and engineering undergraduates, postgraduate students in ground engineering programmes, mine managers, mine site ground control officers and geotechnical engineers, consultants, equipment suppliers, risk managers and the legal profession. Where appropriate, readers are directed to sources of more detailed or specialist knowledge.

Chapter 1 defines ground engineering and provides an overview of the mine design process and the framework for risk management. After introducing basic coal mining systems and associated terminology, Chap. 2 presents the fundamental physical and applied mechanics principles that underpin ground engineering in general and not just in underground coal mining. These principles are applied and developed further in the next three chapters by considering how the rock mass responds, firstly, to the formation of a single excavation (Chap. 3), then to formation of pillar systems as a consequence of forming multiple excavations (Chap. 4), followed by consideration of interactions between mine workings in the same seam and in adjacent seams (Chap. 5).

Inevitably, the rock mass needs to be supported and reinforced around the perimeter of excavations in order to improve its internal load carrying capacity, to restrict convergence at the mining horizon and to prevent falls of ground. A review of ground support and reinforcement systems, the mechanics of their behaviour and the manner in which they modify rock mass response is presented in Chap. 6. This and the principles developed in earlier chapters provide the basis for reviewing a number of design approaches and options for ground support in Chap. 7.

Chapters 8 and 9 are concerned, respectively, with ground control principles and practices relating specifically to pillar extraction and to longwall mining. Principles and practices relating to bord and pillar mining layouts are encompassed in earlier chapters, particularly Chap. 4 which deals with coal pillar systems.

A range of hazards are common to all forms of underground coal mining and these are addressed in Chaps. 10 and 11. Chapter 10 is confined specifically to the effects, impacts and consequences of ground movement, or subsidence, on the interburden between mine workings and the surface and on the surface. Chapter 11 presents a wide range of other hazards and emphasises the need for a cross-disciplinary approach when addressing some of these.

Throughout these first 11 chapters, reference is made regularly to elements of risk management. The text concludes by bringing the entire ground engineering process and its management together in Chap. 12 under a risk management framework. Ground Control Management Plans (GCMPs) give effect to the risk management process. The generic structure of a GCMP is presented and supported with six appendices of associated information. Extracts from actual GCMPs are presented in both Chap. 12 and some of the appendices. This includes examples of procedures required to support a GCMP, such as Trigger Action Response Plans (TARPs) and a Change Management procedure. The chapter concludes with a review of aspects of instrumentation and monitoring essential to monitoring for effectiveness and change and to responding in an appropriate and timely manner to variances from planned performance.

This text deliberately does not suggest the use of specific design procedures. There are a number of fundamental reasons for taking this approach. Some of the more important are, firstly, there are few, if any, design procedures that are entirely accurate or that apply to all circumstances. Secondly, a range of design approaches to a problem are often available. Thirdly, ground engineering is an evolving discipline and not only may better design procedures evolve in time to come, but some that are considered acceptable today may subsequently be found to be flawed or to have additional limitations. Fourthly, the reader is encouraged to understand and to critically evaluate the relevance and reliability of design approaches for themself, consistent with the philosophy of risk assessment. In some cases, this may require seeking third-party advice.

In all cases, critical designs should be subjected to peer review as part of the risk assessment process. Notwithstanding this, aspects of a number of design procedures have been discussed to help the end-user to better understand the degree of confidence to be placed in them and in identifying the types of controls and contingencies that may need to be implemented to manage unplanned outcomes. These aspects all reflect the opening statement in that ground engineering is characterised by pervasive uncertainty.

It cannot be over emphasised that, first and foremost, the moral and professional responsibility of those involved in ground engineering is to safeguard the health and safety of mine personnel and the general public. The most important measure of sound ground engineering is that everyone returns home from work safe and well.

Manly, NSW, Australia J.M. Galvin
December 2015

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A wide range of historical material was offered to the author to support the writing of this text. This was complemented with the author's archive of photographs, plans and reports collected over four decades. Every endeavour has been made to identify the original owner of this information and to obtain their permission to reproduce it. This was not always possible, however, and the author apologies for any oversight. A high reliance has been placed on photographs since a picture is worth a thousand words, especially when dealing with underground environments. Some pictures depict a good situation but others do not. Therefore, unless specifically requested to do so, the source and location of many pictures have not been identified in the text, albeit permission had been received to republish them.

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# Scope of Ground Engineering

# 1

## Abstract

Ground engineering is concerned with the design, construction, operation, maintenance and, ultimately, the closure of safe, serviceable, durable, environmentally sustainable and economic structures built on or within geological materials. It is characterised by pervasive uncertainty and, therefore, needs to be practiced within a risk management framework.

This first chapter notes the wide range of professional competencies involved in ground engineering and some of the models proposed in attempting to clarify interactions between the underpinning and, sometimes, competing disciplines. The three geo-engineering triangles of the practice are defined, being engineering geology; geomechanics; and geotechnical engineering. It is shown how these provide a basis for developing geological models, turning them into ground models and, ultimately, geotechnical models.

The design process in ground engineering is quite different to that in most other branches of engineering. Factors that contribute to this situation in general practice and, more specifically, in an underground environment are presented. The state of the art is reviewed and supported by an appendix that provides a brief history of key developments in ground engineering relevant to underground coal mining.

The chapter concludes by presenting the basic framework for managing risk as a basis for discussing ground engineering in a risk management context in subsequent chapters, with the final chapter dealing with risk based ground management systems in more detail. Evidence is presented of the positive impact that a risk management approach supported by technological innovation has had on the safety and productivity performance of the underground coal mining sector in Australia.

## Keywords

Bow tie analysis • Engineering geology • Fatality rate • Geological model • Geomechanics • Geotechnical competencies • Geotechnical engineering • Geotechnical model • Ground engineering • Ground

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model • History of ground control • History of mining • Lost time injury frequency rate • Mining domains • Risk management

## 1.1 What Is Ground Engineering

Ground engineering is concerned with the design, construction, operation, maintenance and, ultimately, the closure of safe, serviceable, durable, environmentally sustainable and economic structures built on or within geological materials. The practice relies on a range of interdisciplinary and interdependent strands, with distinctions between the various professional competencies and roles being somewhat clouded and ambiguous. This is due to terminology evolving in a fairly loose manner and to overlap in geoscience and engineering disciplines and the competencies and interests of practitioners in these fields.

The range of professional competencies associated with ground engineering is reflected in the number of learned societies that represent the interests of practitioners in this field. The three principal global societies are: (1) The International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE); (2) The International Society for Rock Mechanics (ISRM); and (3) The International Association for Engineering Geology and the Environment (IAEG).

The term ‘ground engineering’ has been adopted in this text because it embraces all aspects of the environments in which coal mines are constructed and operated. Consideration was given to the term ‘geotechnical engineering’ but this option was not pursued because ground engineering in coal mining embraces a wider scope of issues than many associate with geotechnical engineering.

A number of models have been advanced in attempts to illustrate and clarify the interactions of the various disciplines that contribute to engineering in geological materials. One of the more widely acknowledged is shown in Fig. 1.1. Morgenstern (2000) developed this model and used it as a basis for advocating that major value added contributions arise from an integrated or holistic approach to geotechnical engineering.

Morgenstern’s model evolved out of that of Anonymous (1999), who advocated a single learned society for geotechnical engineering in the UK. The model suggests that the main deliverables of geotechnical engineering are structural support systems; fluid control systems, underground geo-structures; surface geo-structures; and ground improvement. Morgenstern was of the view that the organisation of the geotechnical community under the banner of the ISSMGE, ISRM and IAEG was not adequate to foster this approach and proposed the formation of a so-called International Geotechnical Union to promote unification, as opposed to specialisation. This need for unification was highlighted to exist not only in geotechnical practice but also in associated educational and research programs.

Knill (2002) proposed a two hub model in which engineering geology precedes geotechnical engineering. The 2003 Joint European Working Group of the IAEG, the ISRM and the ISSMGE modified and extended this model to a three hub model by including the discipline of geomechanics. The model has been modified further for the purpose of this text; primarily by including the important risk management and continuous improvement feedback loops of ‘monitoring for effectiveness’ and ‘monitoring for change’ and embedding all elements of the ground engineering process within a risk management framework, as shown in Fig. 1.2. These latest modifications are in recognition of the need to manage uncertainty associated with all aspects of ground engineering.

The three geo-engineering triangles of ground engineering shown in Fig. 1.2, being engineering geology, geomechanics and geotechnical engineering, are defined in Table 1.1. Associated

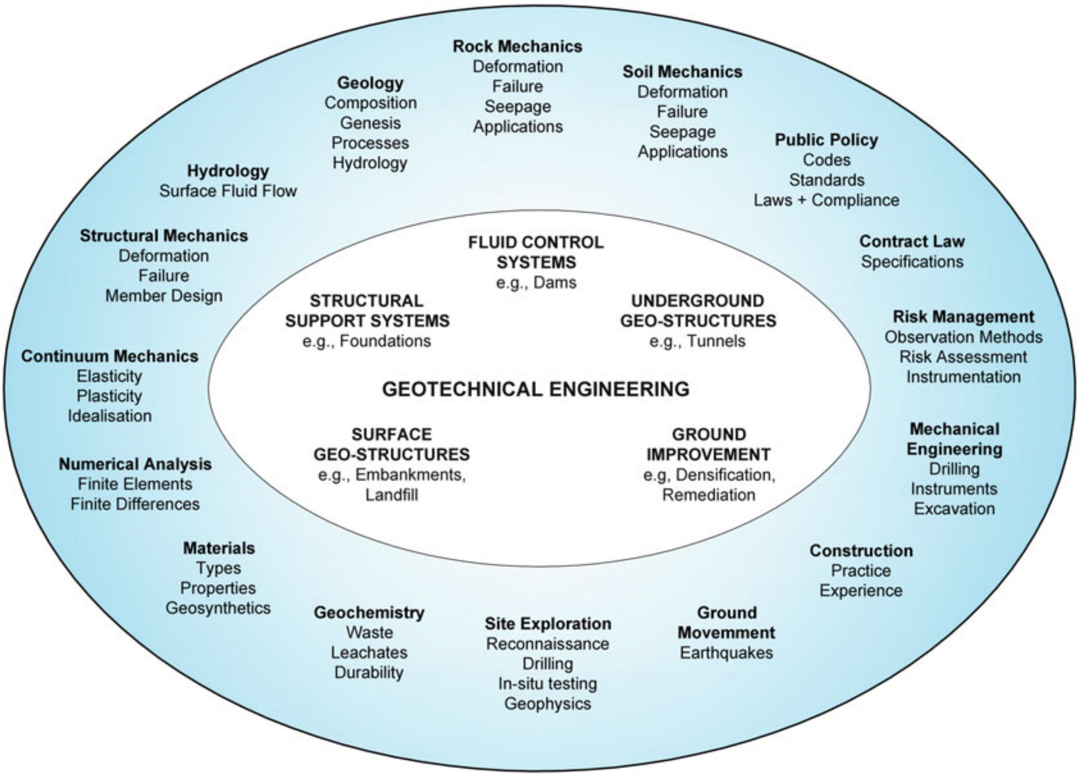
Rock Mechanics
Deformation
Failure
Seepage
Applications
Geology
Composition
Genesis
Processes
Hydrology
Soil Mechanics
Deformation
Failure
Seepage
Applications
Public Policy
Codes
Standards
Laws + Compliance
Hydrology
Surface Fluid Flow
Structural Mechanics
Deformation
Failure
Member Design
Contract Law
Specifications
Risk Management
Observation Methods
Risk Assessment
Instrumentation
Continuum Mechanics
Elasticity
Plasticity
Idealisation
Numerical Analysis
Finite Elements
Finite Differences
Mechanical Engineering
Drilling
Instruments
Excavation
Materials
Types
Properties
Geosynthetics
Construction
Practice
Experience
Geochemistry
Waste
Leachates
Durability
Site Exploration
Reconnaissance
Drilling
In-situ testing
Geophysics
Ground Movement
Earthquakes
FLUID CONTROL SYSTEMS
e.g., Dams
STRUCTURAL SUPPORT SYSTEMS
e.g., Foundations
UNDERGROUND GEO-STRUCTURES
e.g., Tunnels
GEOTECHNICAL ENGINEERING
SURFACE GEO-STRUCTURES
e.g., Embankments, Landfill
GROUND IMPROVEMENT
e.g, Densification, Remediation

**Fig. 1.1** Model developed by Morgenstern (2000) which illustrates the wide variety of professionals and skills involved in the practice of geotechnical engineering

disciplines include geology, hydrogeology, soil mechanics, rock mechanics, applied mechanics, mining engineering and civil engineering. The structures and relationships shown in Fig. 1.2 are particularly apt to ground engineering and management in underground coal mining and have been adopted in this text. However, there continues to be a lack of clarity around the roles and responsibilities of practitioners engaged in ground engineering and efforts are ongoing to resolve this, as reflected for example in JEWG (2008) and Turner (2011).

Based on Fig. 1.2, the first stage in engineering a structure in natural ground is to utilise engineering geology to develop a **geological model** from observational and site investigations. This model is then developed into a **ground model** by embedding the geomechanical engineering parameters required for subsequent design to produce the **geotechnical model**. These parameters can be categorised as intact material properties; rock mass fabric and properties; and environmental influences, such as in-situ stress, groundwater and gas. The geotechnical model forms the basis for evaluating the stability of a ground structure throughout each stage of its life cycle. Since the structure might have to satisfy a range of performance criteria, both simultaneously and over its life, a number of design and assessment iterations employing a number of different analytical and numerical techniques might be associated with the ground model.

It is common to break large structures such as mines into structural regions, or **domains**, based on an expectation that the rock within a domain will have a characteristic quality and behaviour and, in some instances, failure mode. The selection of appropriate domains and geotechnical models of behaviour is critical to the production of sensible designs and stability assessments.

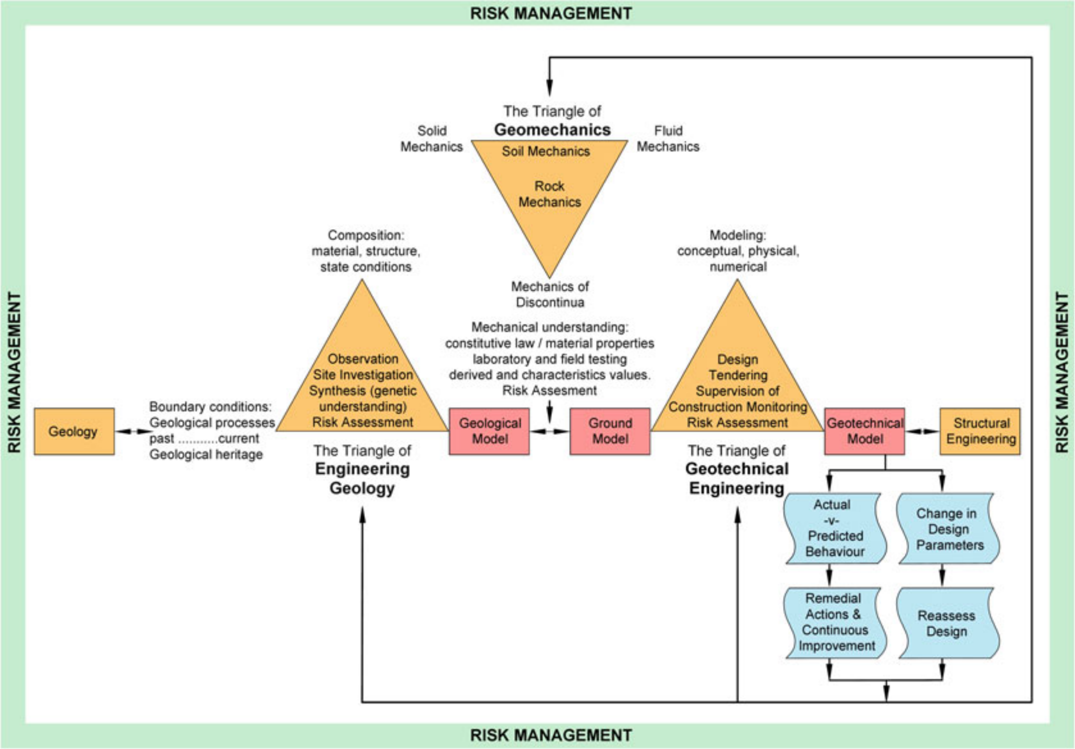
RISK MANAGEMENT
The Triangle of
Geomechanics
Solid Mechanics
Fluid Mechanics
Soil Mechanics
Rock Mechanics
Mechanics of Discontinua
Composition: material, structure, state conditions
Modeling: conceptual, physical, numerical
Mechanical understanding: constitutive law / material properties laboratory and field testing derived and characteristics values. Risk Assesment
Observation
Site Investigation
Synthesis (genetic understanding)
Risk Assessment
Design
Tendering
Supervision of Construction Monitoring
Risk Assessment
Geology
Boundary conditions: Geological processes past ...........current Geological heritage
Geological Model
Ground Model
Geotechnical Model
Structural Engineering
The Triangle of Engineering Geology
The Triangle of Geotechnical Engineering
Actual -v- Predicted Behaviour
Change in Design Parameters
Remedial Actions & Continuous Improvement
Reassess Design
RISK MANAGEMENT
RISK MANAGEMENT
RISK MANAGEMENT

**Fig. 1.2** A ground engineering process model, showing multidisciplinary and interdisciplinary interactions embedded within a risk management framework (Modified and extended from Bock et al. 2004)

**Table 1.1** Elements involved in applying geology, mechanics and engineering to ground engineering (After Galvin 2008)

| 'Geo' discipline | Description |
|---|---|
| Engineering geology | The interface between the observation and description of natural processes associated with the science of geology and the knowledge of numeracy and material properties required for the design and manufacturing central to the engineering process (Knill 2002). It involves observation and site investigation to establish ground composition as a basis for constructing a geological model. Soils and rocks have to be classified according to factors such as their geological age, material type, distribution, structure and state of stress. Physical and chemical properties and the distributions and concentrations of fluids (liquid and gaseous) also have to be determined. |
| Geomechanics | The study of the physical and mechanical properties and response of geological materials and their interactions with water. It embraces soil mechanics and rock mechanics and provides the basis for developing a mechanical understanding of material properties and relevant constitutive (deformation) laws. This information is used to develop the geological model into a ground behaviour model. |
| Geotechnical engineering | Concerned with the design and construction phases and involves analysing and predicting rock mass behaviour. This is done by subjecting the ground model to a series of geotechnical modelling investigations to determine the primary variables governing structural performance; undertaking sensitivity analysis of deficiencies and errors in base information; and evaluating the strength of the structure against the loads imposed on it in both time and space. In underground mining, the structure usually comprises a system of excavations and pillars. |

## 1.2 Peculiarities of Ground Engineering

In most engineering disciplines, design is focused on preventing structural failure; the designer can select the materials used to construct the structure; specifications can be nominated for these materials; the quality of the materials can be controlled during manufacture; and the material properties can be validated and defined with reasonable precision prior to use in the construction.

The situation is quite different in ground engineering, due predominantly to the geological nature and setting of the construction materials with which the engineer has to work. In particular, geological materials:

- are natural and, therefore, not subjected to quality control during their ‘manufacture’;
- have material and mechanical properties which are scale dependent in both space and time;
- are often heterogeneous (not uniform in composition);
- are usually anisotropic (having different properties in different directions);
- often comprise discontinuous fractured media (such as joints, fractures, bedding planes, faults);
- might be subjected to complex solid-fluid interactions.

The situation is more complex in underground mining because:

- forces can be orders of magnitude greater than in other ground engineering situations;
- variations in material properties and the presence of natural material defects are inevitable given the scale of operations;
- economics and practical considerations do not permit the extent and density of site characterisation that occurs in other ground engineering sectors;
- the inflexibility of mine layouts and extraction systems makes it almost inevitable that poor quality materials cannot readily be avoided;
- uncertainties are associated with the magnitude and direction of pre-mining loads within the rock mass;
- uncertainties are associated with design loads and field strengths;
- mines are large, dynamic environments and so stress environments, structural loads and strengths change over an extended period of time as a mine expands and changes shape;
- construction and/or operation may involve materials that are already in a failed state or that will be deliberately brought to a failed state on a local or even regional scale during the mining process;
- structural behaviour can be influenced by far-field factors;
- the financial value of the activity resides in the excavated material and not in the formation of a long term stable void, therefore often resulting in very large excavations that may be intended to have a minimal operational life.

A further complicating factor in mining is that optimum geotechnical designs may need to be compromised in order to accommodate practical aspects of mining, and vice versa. For example, the shape of a mining lease, the management of water inflow, or the control of gas migration down-dip or up-dip (depending on its density relative to air) may individually or collectively result in a sub-optimum mine layout from a ground control perspective in a high in situ horizontal stress environment. Similarly, quality considerations may prevent coal from being extracted to a roof ply that maximises natural support capacity. Competing and conflicting demands may also arise in circumstances such as multiseam mining. The mining engineer, therefore, has to have a close involvement in the ground engineering process in order for it to deliver safe, practical and economic design outcomes.

## 1.3 State of The Art

A brief history of the key developments in ground engineering is presented in Appendix 1. Up until the mid 1950s, the approach to ground control in underground coal mining was largely empirical and trial and error based. The development of rock mechanics as a scientific discipline since that time, in conjunction with advances in field instrumentation and monitoring, ground support technologies, and computer based numerical analysis has led to the evolution of ground engineering. The more important theoretical and applied developments relevant to underground coal mining during this period include:

- Recognition that the mode of in situ rock failure is controlled by both the properties of the rock itself and by the load-deformation characteristics of the surrounding rock mass, or loading system. This, in turn, has led to a mechanistic understanding of controlled and uncontrolled rock failure and recognition that rock still maintains a substantial load carrying capacity after being loaded beyond its point of peak stress, or strength.
- Recognition that the load-deformation behaviour of a rock mass beyond the fractured skin of an excavation can be simulated approximately by a linear elastic model.
- Advances in computational power and developments in numerical modelling software codes, thereby enabling increasingly complex mining situations to be simulated.
- Developments in understanding the mechanics of blocky rock masses, in methods of analysis for blocky jointed rock, and in applying outcomes to excavation engineering and support and reinforcement design.
- Developments in instrumentation and monitoring techniques, most notably related to measuring in-situ stress and rock mass displacement and to microseismic monitoring to detect the location and magnitude of fracturing deep within the rock mass. This insight enables classical mechanics to be invoked to explain ground behaviour.
- Microprocessor monitoring of instrumentation and mining equipment to provide continuous and real time information on ground response.
- Technologies for internally reinforcing the rock mass to improve its self-supporting capacity.
- Static and kinematic configuration, control and monitoring of hydraulic powered supports on longwall faces.
- Digital wireless technology for automation of data transmission.

Nevertheless, considerable uncertainty is still associated with ground engineering in a mining environment. In the first instance, there are serious limitations with site characterisation. For example, a typical 500 m grid pattern of cored boreholes for initially laying out an underground coal mine only provides for physically sampling and observing some 0.000003 % of the rock mass.

Furthermore, the geological and environmental factors that contribute to the ground model cannot be portrayed in engineering terms by single numbers. As noted by Knill (2002), variability is a consequential characteristic of geological processes, and a particular engineering property is likely to be represented by a distribution of values derived from laboratory and field tests. Such a property is most simply represented within the ground model by a single operational value, which may be selected by a variety of methods, including precedent experience, a judgement process or some form of probabilistic parametric analysis. Outcomes can be variable and their reliability is highly dependent on the skill and experience of the individuals undertaking the investigations and analysis. Conclusions have to rely upon extrapolation, deduction, inference, statistics and probability, intuition and experience.

Although rock mechanics and its application to ground control have evolved rapidly since the early 1950s, there are still many deficiencies in the knowledge base. For example, Hudson and Harrison (1997) noted in relation to numerical modelling that for a fully anisotropic rock, 21 (material) constants are needed and to their knowledge, these 21 constants have never been measured in a rock engineering project. Almost

20 years later, it is still not practical to measure these constants on a routine basis. Hence, establishing the balance between not including enough rock property information and conducting unnecessary complex analysis is difficult. It is rare for there to be one universally acceptable or exact design approach and decisions may ultimately need to rely on making judgements between competing options.

It is inevitable, therefore, that design outcomes are imprecise. In addition to inadequacies and errors in data collection and site characterisation, simplifications have to be made in progressing from a geological model to a ground model and then to a geotechnical model. Design outcomes are also vulnerable to change over time, especially in underground mining where natural and artificial ground support and reinforcement systems may have a finite lifespan; the behaviour of strata in and surrounding a mine can be time dependent; and external factors may arise in the long term that were unforeseeable at the time of design.

Consequently, ground engineering is characterised by pervasive uncertainty. This uncertainty is present throughout the full life cycle of a mine, from site characterisation, through design, construction, daily operation, and for an indefinite period after mine closure and abandonment.

## 1.4 Risk Management

Uncertainty gives rise to **risk**, which is a combined measure of the **likelihood** of an event occurring and the **consequences** should the event occur. A source of potential harm or loss which could result in an event is referred as a **hazard**, with any means by which the hazard could materialise being referred to as a **threat**. The consequences of an event influence the level of uncertainty that can be tolerated. In the case of ground control, a loss of stability can have implications for safety, environment, economic performance, exposure to litigation, and corporate reputation. These consequences can be particularly high in underground coal mining and so the tolerable likelihood of failure in this sector is correspondingly low. Although ground engineering can impact on the full spectrum of enterprise risks, the first and foremost responsibility of those involved in the practice is to ensure that their activities do not threaten the health and safety of employees or the public. Ground control practitioners have moral, statutory and professional responsibilities in this regard. A safety focussed risk assessment should always be the first step in testing a design concept.

Traditionally, a high reliance has been placed on prescriptive legislative to achieve a safe place of work. However, experience has shown this approach to have been only partially effective, with the Robens Inquiry (Robens et al. 1972) concluding that this is largely because this type of legislation:

- is reactive, often having been framed in response to an incident;
- does not cover all circumstances;
- does not keep up to date with evolving knowledge, technology and practice;
- does not encourage owners and management to seek out risks and develop their own set of controls;
- can create a mindset amongst management that, because they are complying with the prescribed legislative requirements, health and safety are being properly managed.

The Robens Inquiry recommended a shift from prescriptive occupational health and safety legislation to so-called ‘enabling’ legislation, in which the regulator sets the performance standards and leaves it up to the owner of the risk to decide how to best achieve these standards. This recommendation has been implemented in many countries whose legal system is based on the British system. It is founded on formalised risk management processes that

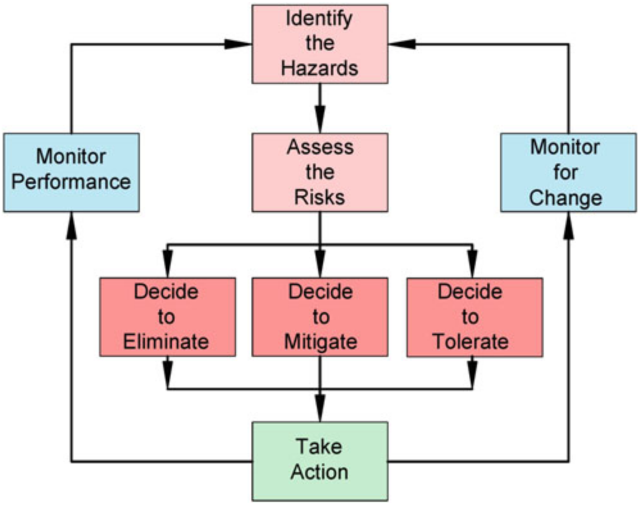

**Fig. 1.3** The basic framework for managing risk (After Joy 1998)

have regard to *ISO 31000:2009 Risk Management – Principles and Guidelines* (ISO 31000 2009), an international standard that applies to controlling all forms of risk, not just risk to health and safety. These processes have been adopted by the International Labour Organisation in its *Safety and Health in Mines Convention, 1995 (No 176)*.

Fig. 1.3 summarises the fundamental steps involved in the risk management process. Consistent with ISO 31000, 'consultation' and 'communication' need to be embedded in all elements of this process. The process commences with identifying hazards and then assessing associated likelihood and consequence in order to determine the risk presented by each hazard. Next, controls are devised to eliminate each hazard where possible, or otherwise to reduce the risk associated with it to an acceptable level. These controls need to be risk assessed in their own right to confirm their likely effectiveness, to verify that they will not give rise to higher risks than those they are intended to address, and to determine residual risk levels. Then, having implemented the controls, it is essential that performance is monitored to verify the effectiveness of the risk assessment process. It is also essential that monitoring for change is undertaken to identify any deviations from the conditions and circumstances on which the risk management process was based and to intervene in a timely manner before a hazard materialises. Hence, the risk management framework equates to a continuous improvement process of Plan-Do-Check-Act.

Risk management has added significance for ground engineering in an underground mining environment because 'recovery measures' may also need to be pre-planned to minimise the consequences of any ground instability. A so-called **bow tie diagram**, illustrated in Fig. 1.4, is a particularly useful risk management tool for analysing risk. It provides a powerful graphical representation of upstream threats and downstream consequences and facilitates the identification of preventative controls and, should the unwanted event still occur, contingencies for mitigating the consequences. Bow tie analysis finds extensive application in ground engineering in those regimes operating under a risk management framework (discussed in more detail in Chap. 12).

## 1.5 The Impact of Risk Management and Technology

The Australian mining industry began to adopt a risk management approach to health and safety in

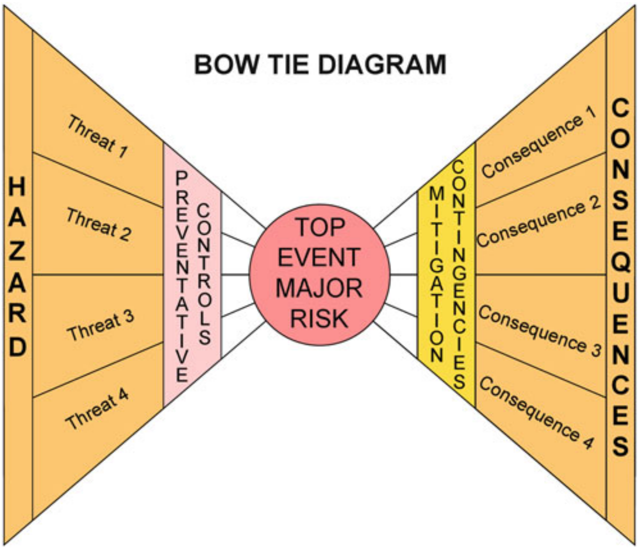

**Fig. 1.4** The concept of a ‘bow tie’ diagram for analysing risk

the mid 1980s and to support it with a range of guidelines and research and development initiatives. Two notable risk management guidelines which remain relevant are *MDG 1010: Risk Management Handbook for the Mining Industry* (MDG-1010 1997 – republished in 2011) and *MDG 1014: Guide to Reviewing a Risk Assessment of Mine Equipment and Operations* (MDG-1014 1997).

The identification of hazards and the need to then reduce risk associated with these hazards to acceptable levels is a major driver of innovation. Many of the ground engineering controls that are taken for granted today were either not available in the 1980s or in a very early stage of development. This is reflected in Fig. 1.5, which is an abridged version of new technology requirements for improving safety, productivity and costs in the Australian underground black coal mining sector as identified by McCarthy (1987). The green (shadowed) boxes indicate topics that are now well advanced as a result of subsequent research.

The benefits of a risk management approach supported by technological innovation are reflected in trends in the safety performance of the NSW coal sector and the productivity of the Australian black coal mining sector, some of which are shown in Figs. 1.6, 1.7, and 1.8. This sector produced just over 74 million tonnes of raw coal from underground mines and almost 187 million tonnes from surface mines in FY2013-14 (July to June). Since the early 1980s, the sector has experienced a 15 fold decrease in fatalities, with a number of fatality free years, and a tenfold decrease in lost time injuries per one million employee hours worked, or LTIFR (Fig. 1.6). Improvements have been particularly pronounced in ground control, with ground instability related incidents accounting for only four fatalities in the 13 years to 2014 and only 2 % of all injury compensation claims as at 2007 (Fig. 1.7), down from 16 % in 1995 (later comparisons are restricted by changes in data recording).

The significant improvements in health and safety have been accompanied by significant improvements in productivity, shown in Fig. 1.8. This is consistent with the adage that ‘a safe mine is a productive mine’. It reflects the discipline that risk management systems supported by well-engineered controls bring to controlling all risks associated with ground control; not just those related to health and safety. For these reasons, risk management and research developments are given high priority in this text.

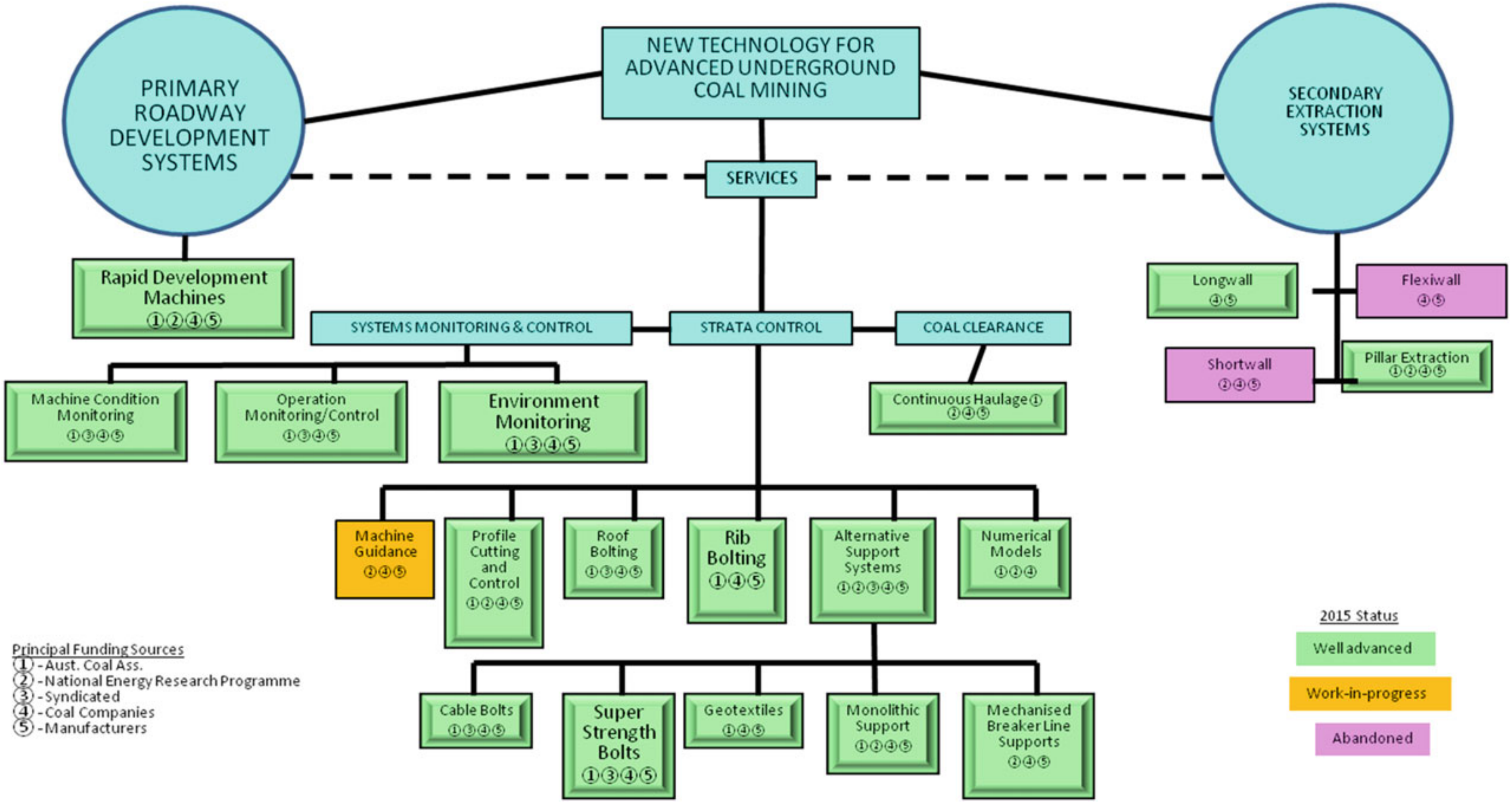
PRIMARY ROADWAY DEVELOPMENT SYSTEMS
NEW TECHNOLOGY FOR ADVANCED UNDERGROUND COAL MINING
SECONDARY EXTRACTION SYSTEMS
SERVICES
Rapid Development Machines ①②④⑤
SYSTEMS MONITORING & CONTROL
STRATA CONTROL
COAL CLEARANCE
Longwall ④⑤
Flexiwall ④⑤
Shortwall ②④⑤
Pillar Extraction ①②④⑤
Machine Condition Monitoring ①③④⑤
Operation Monitoring/Control ①③④⑤
Environment Monitoring ①③④⑤
Continuous Haulage ① ②④⑤
Machine Guidance ②④⑤
Profile Cutting and Control ①②④⑤
Roof Bolting ①③④⑤
Rib Bolting ①④⑤
Alternative Support Systems ①②③④⑤
Numerical Models ①②④
Cable Bolts ①③④⑤
Super Strength Bolts ①③④⑤
Geotextiles ①④⑤
Monolithic Support ①②④⑤
Mechanised Breaker Line Supports ②④⑤
Principal Funding Sources
① - Aust. Coal Ass.
② - National Energy Research Programme
③ - Syndicated
④ - Coal Companies
⑤ - Manufacturers
2015 Status
Well advanced
Work-in-progress
Abandoned

**Fig. 1.5** An abridged version of the technology requirements identified by McCarthy (1987) for improving safety, productivity and costs in the Australian underground black coal sector and their status in 2015

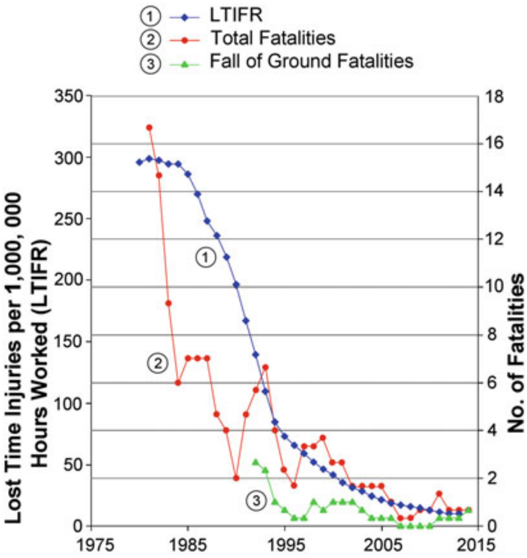

**Fig. 1.6** Trends in safety performance measures for the NSW coal mining sector, expressed as three year rolling averages (Plots based on data compiled by the Joint Coal Board and by Coal Services Pty Limited)

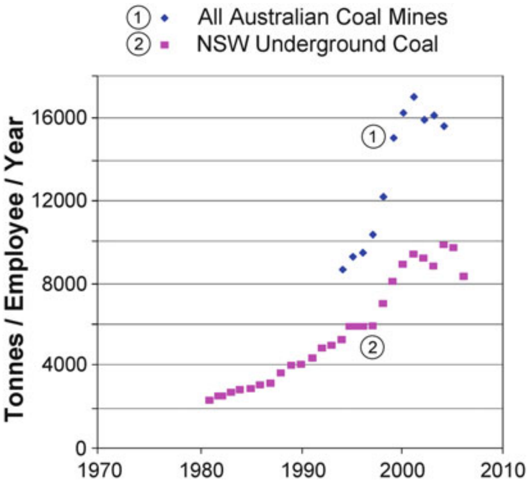

**Fig. 1.8** Plots of productivity of NSW and Australian coal mines in the period 1980 to 2006, whereafter the reporting system changed (Plots based on data compiled by the Joint Coal Board and by Coal Services Pty Limited)

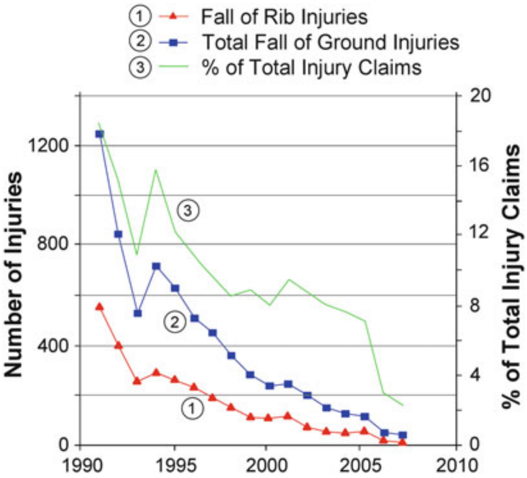

**Fig. 1.7** Trends in injuries due to all types of falls of ground and to falls of rib only, expressed as a percentage of the total number of injuries in the NSW coal sector (Plots based on data compiled by the Joint Coal Board and by Coal Services Pty Limited)

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# Fundamental Principles for Ground Engineering

# 2

## Abstract

In essence, all underground mining methods are the same, comprising one or more excavations separated by pillars of rock sandwiched at some orientation between the hanging wall and the footwall. The progressive removal of rock to form an underground excavation results in a decrease in the load carrying capacity of the immediate surrounding rock mass, the creation of a void into which the rock mass can displace, and stress changes in a rock mass that is weakened by the removal of confinement. The resulting rock deformation is governed by both the structural and the mechanical properties of the rock mass and the surrounding stress environment.

This chapter commences with an overview of the geological settings of underground coal mines and of the generic types of mining techniques and mine layouts utilised in the given conditions. This provides context to the basic concepts of physics and applied mechanics that control rock deformation and underpin ground engineering. The more fundamental of these are developed from first principles in the remainder of the chapter under the headings of rock mass fabric; physical parameters; material properties; rock mechanics; analysis techniques; and statics. This provides a foundation for developing the principles in more detail and applying them to mine design, stability analysis, operational practices and risk management in later chapters. Their application is not confined to underground coal mining, with many being applicable to any form of excavation made in a geological setting.

## Keywords

Analytical models • Bayes' rule • Bord and pillar mining • Boundary element method • Classical beam theory • Coal mine design • Coal mining methods • Constitutive laws • Domain methods • Dynamics • Effective stress • Elastic theory • Empirical models • Equivalent modulus • Euler buckling • Failure criterion • Failure modes • Field stress • Flexural rigidity • Geological Strength Index • Ground response curve • Hoek Brown failure criterion • In situ stress • Kinematics • Linear arch theory • Longwall mining • Mohr Coulomb failure criterion • Numerical

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modelling • Plate theory • Primitive stress • Probabilistic analysis • Resultant stress • Rock mass classification system • Room and pillar mining • Safety factor • Statics • Statistical analysis • Stochastic analysis • Strata stiffness • Strength of rock • Stress conventions

## 2.1 Introduction

In essence, all underground mining methods are the same, comprising one or more excavations separated by pillars of rock sandwiched at some orientation between the hanging wall and the footwall. The progressive removal of rock to form an underground excavation:

- removes confinement to all surfaces of the excavation, causing a decrease in the load carrying capacity of the immediate surrounding rock mass and the creation of a void into which the rock mass can displace; and
- redirects the stresses originally carried by the removed rock so that they now pass around the excavation, resulting in stress changes in the same surrounding rock mass weakened by the removal of confinement.

Hence, consequential deformation and change in the fabric and mechanical properties of the rock mass may be due to either stress relaxation or overstressing, or to a combination of stress relaxation in one direction and overstressing in another.

Rock deformation around a mining excavation is governed by both the structural and the mechanical properties of the rock mass and the surrounding stress environment. Techniques for investigating, collecting and presenting geological and structural data to construct a geological model are covered comprehensively in other publications. This text is focused on presenting the engineering principles that impact on the safe and efficient underground extraction of coal and applying them to mine design and operating practice. The basic foundation principles are presented in this chapter and are confined primarily to one and two dimensions to facilitate understanding amongst practitioners. To provide context to some of these principles, the chapter commences with an introductory description of mine layouts and an overview of ground conditions associated with underground coal mining.

## 2.2 Characteristics of Underground Coal Mining

### 2.2.1 Geological Setting

Coal is an organic product with no crystalline structure and, therefore, technically it is not a mineral. In many countries, coal is assigned the status of a mineral by legislation. Two common traits of coal seams are that they comprise a series of sub-horizontal plies associated with different phases of their formation, and one or more sub-vertical, orthogonal joint sets that may be ply specific or extend through the full thickness of the seam. These internal joints are referred to as **cleats** and arise from compaction and diagenesis. The two traits impart a characteristic blocky structure to coal seams and give rise to considerable variation in their structural and mechanical properties, both within an individual seam and between seams.

Another characteristic of coal seams is that they form in sedimentary environments. This generally results in the seams being tabular and laterally extensive; to being sandwiched between sedimentary strata that is bedded and of low to moderate strength; and being interspersed with sedimentary bands, or partings. Well developed sub-vertical joint systems may also exist in the surrounding strata. There can be considerable variation between the thickness of strata and significant contrast in the mechanical properties of adjacent strata.

During and subsequent to diagenesis and coalification, the geological setting can also be impacted by tectonic events, such as faulting, folding, uplift and igneous intrusions. These may comprise a series of episodes, with each episode impacting the structure and mechanical properties of the seam in a different manner. They can result in elevated lateral stress in specific directions in a

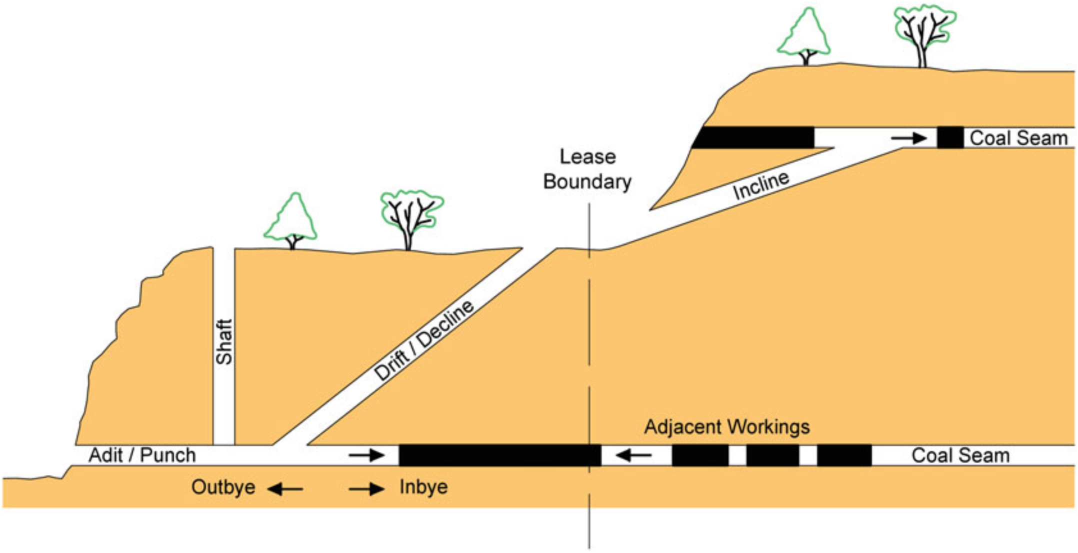


**Fig. 2.1** Common means for accessing an underground coal seam

coal seam and the surrounding strata, with this stress magnitude often being two to three times that of the vertical stress.

It is standard mining practice in flat and shallow dipping seams to align the roof and floor of excavations so that they are parallel to bedding. Hence, excavations are usually rectangular in shape. In conjunction with the structural and mechanical characteristics of the surrounding rock mass, this results in beam, column and plate theory finding application to ground engineering and mine design. In particular, this theory is often applied to the design and assessment of the direction, dimensions and support of excavations within a coal seam and the behaviour of the superincumbent strata.

### 2.2.2 Mine Access

Coal reserves in a mining lease can be accessed in a number of ways, the choice of which is influenced by factors such as topography, depth, number of seams to be extracted and the presence of mine workings in adjacent mine leases. The most common means of access, which may be used in combination, are illustrated in Fig. 2.1 and comprise;

- **Shafts**: Vertical connections between the surface and the working horizon.
- **Drift/Incline/Declines**: Sloping connections (or ramps) between the surface and the mining horizon. The slope or gradient typically ranges from 1 in 10 (or 6°), to permit rubber tyre equipment to operate in the drift, to 1 in 3 (or 18°) when men and materials are transported by rail mounted equipment attached to a rope winder.
- **Adits**: Entries driven in coal from points where the coal seam outcrops on the surface.
- **Highwall punches**: Effectively, a series of adjacent adits driven in the coal seam at the base of an open cut mine.
- Underground connections from existing workings in adjacent leases.

### 2.2.3 Mine Roadways

The general area within a mine from where access is gained to a targeted coal seam is usually referred to as the **pit bottom**. Personnel, materials and services diverge from this point and conveyor belts from the various mining districts report to it, as shown in the example in Fig. 2.2.

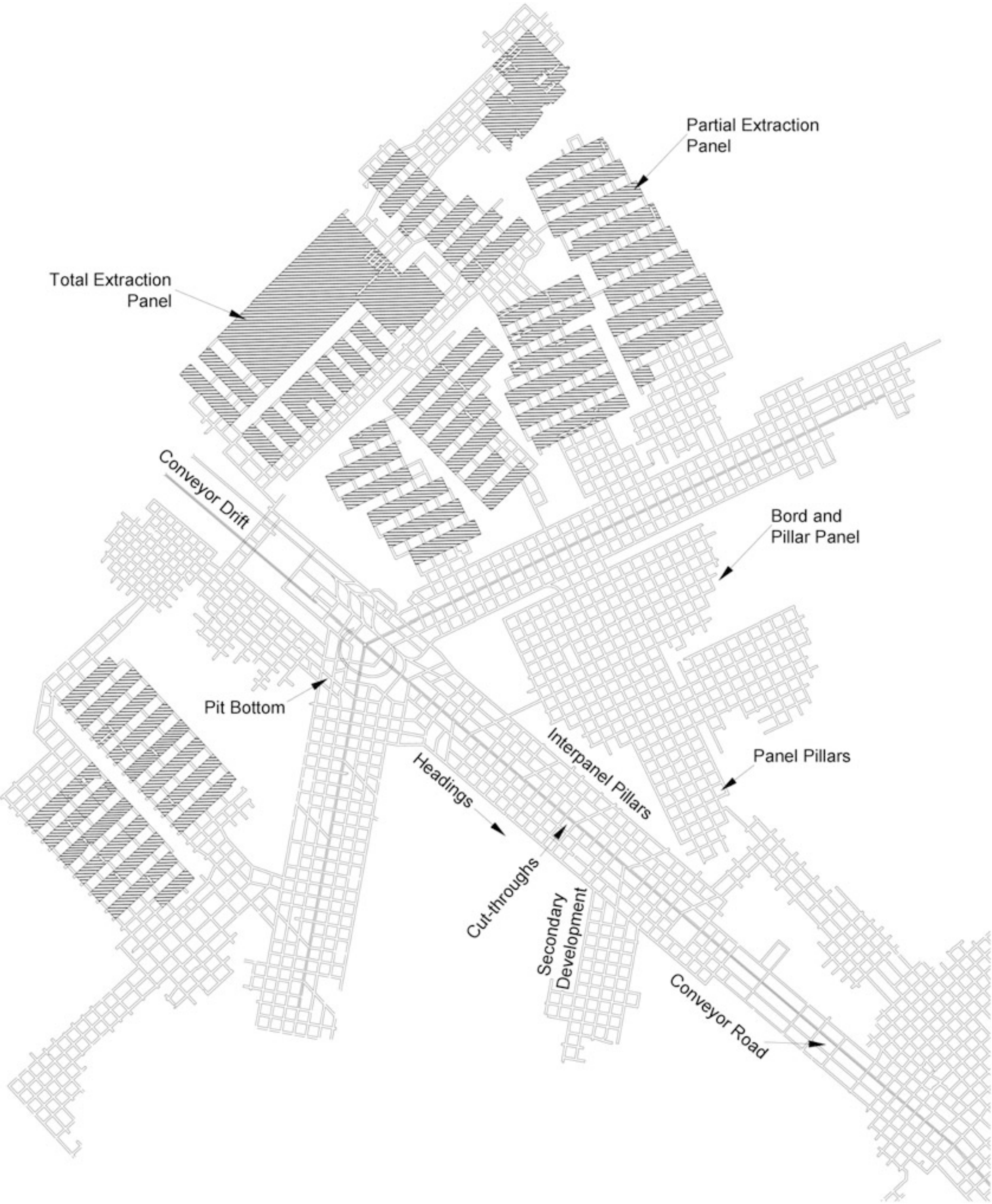

Partial Extraction Panel
Total Extraction Panel
Conveyor Drift
Bord and Pillar Panel
Pit Bottom
Panel Pillars
Interpanel Pillars
Headings
Cut-throughs
Secondary Development
Conveyor Road

**Fig. 2.2** Mining terminology illustrated with the aid of a mine layout designed for bord and pillar mining and selective secondary pillar extraction

Sets of roadways are driven in the coal seam in various directions from the pit bottom to access the coal in the mining lease. These roadways are required to remain in service for an extended period of time, often for the life of the mine, and therefore, are referred to as **main development** roadways.

The **inbye** direction in a mine is the direction of the mining face when approaching from the pit bottom (i.e. going into the mine). The **outbye** direction is the direction of the pit bottom when approaching from the working face (i.e. going out of the mine).

As main development roadways extend further inbye from the pit bottom, additional sets of roadways may have to be driven off them (turned away) in order to access all areas in the mining lease. These new roadways are referred to as either main development roadways or **secondary** development roadways, depending on their expected operational life.

The main source of coal production comes from working districts, referred to as **panels**, which are developed off main development and secondary development roadways. Effectively, extractable coal within a mining lease is 'blocked out' as a series of panels.

Underground roadways in coal mines are referred to generically as **bords** in Australia and South Africa, **rooms** or **entries** in the USA and bords or **stalls** in the UK. Roadways formed in the direction of advance of the main development, secondary development or panels are referred to as **headings**. The spacing between headings typically ranges from 15 to 50 m, centre (of roadway) to centre (of roadway). For practical reasons related primarily to ventilation, materials transport and personnel access and egress, headings are usually connected on a regular basis with roadways referred to as **cut-throughs** or **cross-cuts** as shown in Fig. 2.2. These are usually driven at 60°–90° to the direction of the headings, with spacing typically ranging from 15 to 130 m. It is a common convention within each panel to identify headings numerically or alphabetically from left to right when facing inbye and cut-throughs numerically from outbye to inbye.

The pattern of headings and cut-throughs delineates blocks of coal referred to as **pillars**. Pillars may be square, rectangular or rhomboidal in shape, depending on the angle of the cut-throughs relative to the headings and on the spacing between cut-throughs. Coal pillars within a mining district, or panel, are called **panel pillars**.

It is usually advisable for ground control, ventilation and water control purposes to separate panels by solid ribs of coal, referred to variously as **interpanel pillars**, **barrier pillars**, or **abutment pillars**. These pillars typically range in width from 30 to 100 m. In the final stages of the life of a mine, main developments and secondary developments may also constitute mining panels, with additional coal being extracted from them on retreat out of the mine.

When mechanised mining is employed, roadways are excavated in one of two basic manners, referred to as **cut and bolt** and as **cut and flit**. In the cut and bolt method, the roof is progressively supported at the coal face as the continuous miner advances. The roadway may be cut to full width in one pass or in two passes of the continuous miner. In the cut and flit method, a section of roadway up to 15 m in length is mined, usually in two passes, and then the continuous miner is transferred to another workplace while the newly exposed roof is supported.

### 2.2.4 Mining Methods

Workings which comprise a regular layout of headings and cut-throughs, such as shown in Fig. 2.2, are known as **bord and pillar**, **room and pillar**, or **pillar and stall** workings. This includes main development workings and secondary development workings. When bord and pillar panels are left in an as-formed state they may be referred to as **first workings**. If, subsequently, the coal pillars are partially or totally extracted, the workings are known as **second workings** and the mining operations are referred to as **secondary extraction**.

The process of extracting all pillars in a panel is known as **total pillar extraction**. Sometimes only selected coal pillars in a panel or selective portions of individual coal pillars in a panel are extracted. These operations are referred to as **partial pillar extraction**. An example of a partial pillar extraction practice is that of extracting every second row of pillars, shown in Fig. 2.2. This is commonly referred to as **panel and pillar mining**.

**Longwall mining** is effectively a form of total pillar extraction mining. Longwall blocks, or longwall panels, are typically 150–400 m wide and between 1,500 and 4,000 m long (Fig. 2.3). The **longwall panel** is usually delineated by driving one, two or three roadways down each longitudinal boundary of the block and connecting them at the inbye extremity of the block. The headings comprising the longitudinal roadways are referred to as **gateroads**. The

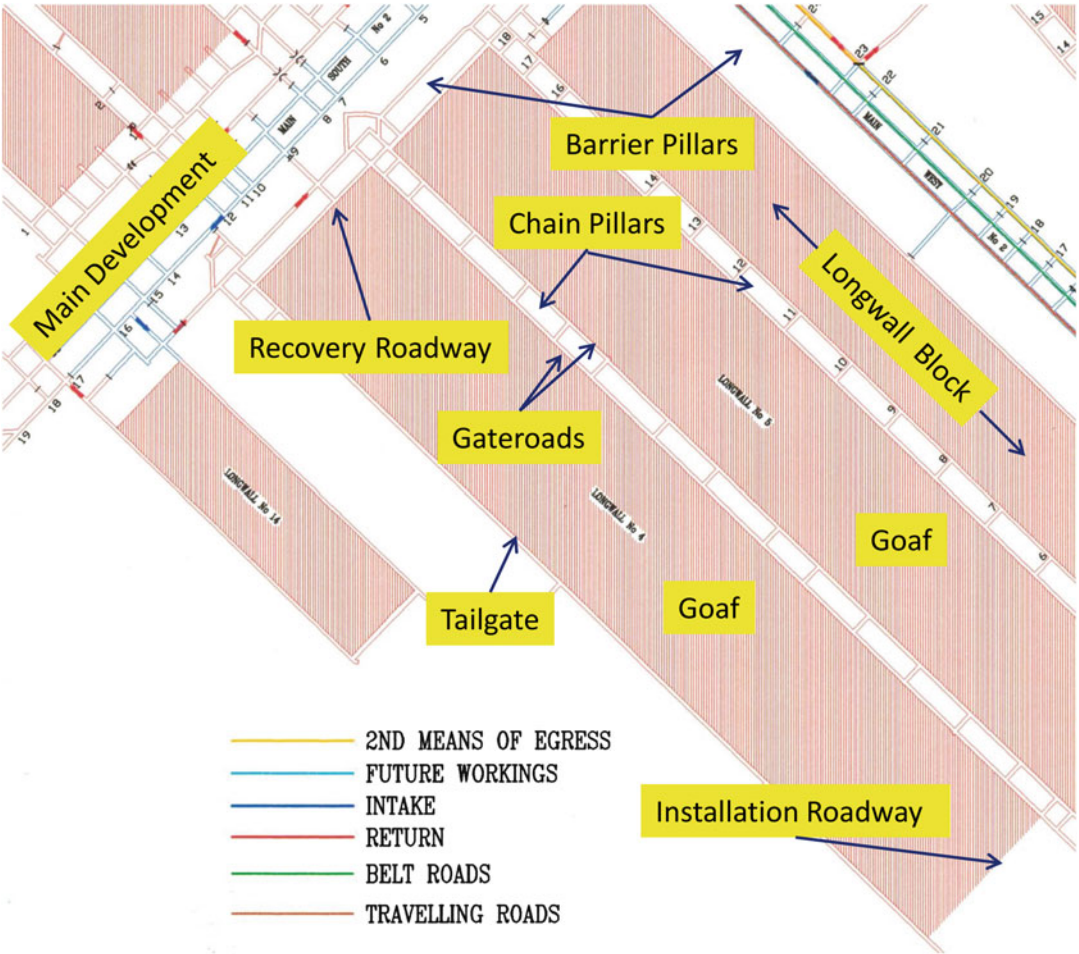

Main Development
Barrier Pillars
Chain Pillars
Recovery Roadway
Longwall Block
Gateroads
Goaf
Goaf
Tailgate
Installation Roadway
LONGWALL No 14
LONGWALL No 4
LONGWALL No 5
MAIN SOUTH
MAIN WEST
2ND MEANS OF EGRESS
FUTURE WORKINGS
INTAKE
RETURN
BELT ROADS
TRAVELLING ROADS

**Fig. 2.3** A mine layout and terminology associated with longwall mining

driving of longwall gateroads is referred to as **longwall development**, with a set of gateroads constituting a **longwall development panel**. Hence, it takes two longwall developments panels to delineate a **longwall block.**

A block of longwall coal is progressively extracted by cutting a series of transverse slices or **webs** of about 1 m width off the end of the pillar, commencing at the extremity of the block (inbye end) and retreating back (outbye) towards the main development. Longwall extraction operations effectively result in the formation of a series of very wide and very long excavations, each separated by one or two rows of relatively narrow pillars. These pillars are referred to as **interpanel pillars** or **chain pillars.**

The worked out area of a pillar extraction panel or a longwall panel is known as the **goaf** in Australia and South Africa and the **gob** in North America. This area may remain open or it may collapse. It is a 'no-go' zone for personnel as a result of being left in an inadequately supported state.

**Highwall mining** involves remotely driving unsupported entries, or punches, for up to 300 m or more into the highwall of an open cut mine using either a continuous miner or an auger. Typically, continuous miner entries are around 3.5 m wide (Fig. 2.4a) and auger entries range from 1.0 to 2.0 m in diameter (Fig. 2.4b). A double row of auger entries can be employed in thicker seams. Entries are separated by panel pillars, or **web pillars**, that have a width-to-height ratio typically in the range of 0.75–1.75. Wider **barrier pillars** are left on a regular basis to define panels. The spacing between these pillars is a function of depth of mining but if

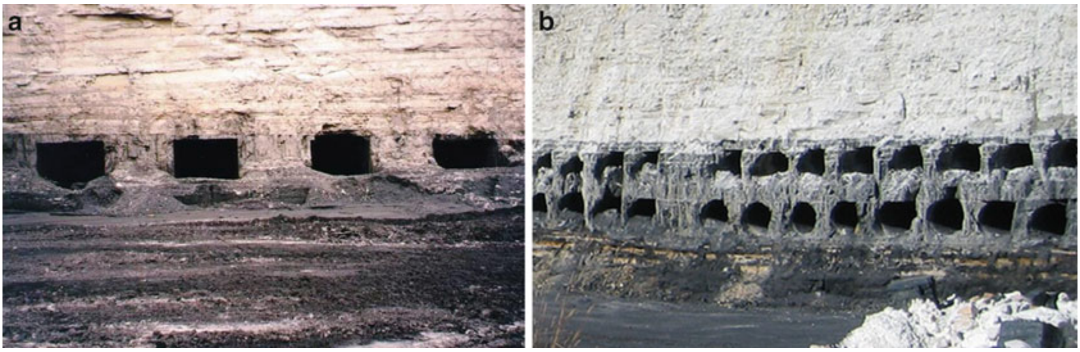


**Fig. 2.4** Examples of highwall mining operations (**a**) Continuous miner plunges (**b**) A double row of auger plunges, with a barrier pillar on the left

poor ground conditions or stability problems are experienced during mining, the spacing may be as little as every four entries. Entries are usually driven normal to the highwall although some mines have experimented with driving entries in sets of three or five with the outer entries having a slight splay so as to form tapered pillars.

Highwall mining is distinguished from standard bord and pillar mining in that the excavations are usually as wide or wider than the pillars and the pillars are very narrow and have a very low width-to-height ratio. Hence, highwall mining equates more with pillar extraction, particularly as the workings are often formed to a relatively low safety factor and are not intended to be permanently stable.

## 2.3 Rock Mass Fabric

A rock mass comprises blocks of intact rock with a range of mechanical properties, delineated by a variety of discontinuities such as bedding planes, joints, foliation, faults and dykes. Rock mass failure is controlled by the formation of new fracture surfaces and by reactivation of movement on existing discontinuities. Hence, ground engineering has to give consideration to both the intact (unfailed) and residual (failed) behaviour of soil and rock and the fluids contained within and around the fabric of these materials in geological settings.

The mode of formation and tectonic history of a rock mass usually impart different structural and mechanical properties in different directions. The description of an element of a rock mass as being intact or fractured is dependent on scale, as illustrated in Fig. 2.5. Table 2.1 lists terminology and meanings ascribed to the more common attributes of rock mass composition.

## 2.4 Physical Parameters

A structure is in equilibrium when the sum of all the forces acting on the structure and the sum of all turning moments generated by these forces are both equal to zero. Due to changes in rock mass strength and stress distribution, strata failure may be an unavoidable part of the process of restoring equilibrium after an excavation has been formed in the rock mass. A primary objective in mine design is to restore equilibrium to a degree that mining can occur without local or regional instability presenting an unacceptable risk to personnel, community, environment and business. In some instances, this may require inducing failure in a controlled manner or controlling failure long enough for mining operations to retreat out of an area.

The laws of physics govern how a body responds when subjected to forces, the most fundamental being Newton’s Three Laws of Motion which are:

- A body will remain in a state of rest or in uniform motion in a straight line unless acted upon by a force.
- When an unbalanced force acts on a body, the body will accelerate.

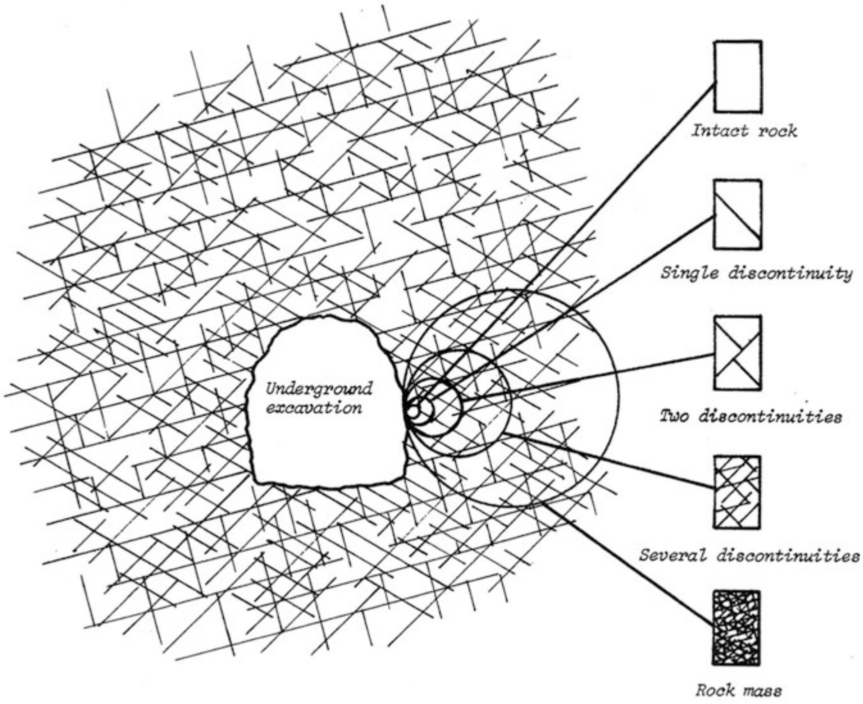


**Fig. 2.5** Idealised diagram showing the transition from intact rock to a heavily jointed rock mass with increasing sample size (After Hoek and Brown 1980, © Taylor and Francis Group)

- If a body A exerts a force on a body B, body B will exert an equal and opposite force on body A.

Measurement systems provide a means for quantifying and describing physical behaviour. Measurement is defined as the process of comparing the quantity of something with some standard reference quantity of the same thing. The most universal set of reference quantities is the *Système Internationale*, SI. The definition of some fundamental physical parameters utilised extensively in rock engineering and their units of measurement are given in Table 2.2.

Forces and displacements which cause a body to shorten are referred to as 'compressive' and those which cause it to stretch or lengthen are referred to as 'tensile'. In rock mechanics, compressive forces predominate and compressive stresses, displacements and strains are regarded as being positive. Ground engineering practitioners need to be alert to the fact that this is the opposite to the convention used in elastic theory, materials science and generally in engineering mechanics (see Sect. 2.6.1).

It is common for very large forces to be present in a mining environment, these being most often associated with the weight of the overlying rock mass and with the hydraulic circuits of equipment. The forces can be visualised as:

| | |
|---|---|
| 1 N | the weight of one apple |
| 1000 N | 1 kN, that is, one kilonewton |
| 1 kN | a weight of 100 kg or 0.1 t |
| 1000 kN | 1 MN, that is, one meganewton |
| 1 MN | a weight of 100 t |
| 1 Pa | the weight (on Earth) of one apple spread over an area of 1 m × 1 m |
| 1 kPa | 0.1 t/$m^2$ |
| 1 MPa | 100 t/$m^2$ |
| 1 GPa | 1000 MPa |

## 2.5 Material Properties

As a mining excavation is advanced, the rock mass about the face of the excavation responds in three dimensions. The analysis of this response can be complex and require sophisticated numerical modelling. If the excavation ends up being

**Table 2.1** Terminology and meanings ascribed to the more common attributes of a rock mass

| | |
|---|---|
| Homogenous | Of uniform composition. Homogeneity is a measure of the physical continuity of the rock mass based on the distribution of discontinuities and pore spaces within the window of interest. |
| Isotropic | Having the same physical properties and, therefore, the same reaction to applied stress in all directions. |
| Transversely isotropic | Having the same physical properties and, therefore, the same reaction to applied stress in two orthogonal directions but not in the third direction. Layered strata, which is a characteristic of most coal sedimentary environments, is often assumed to be transversely isotropic. |
| Anisotropic | Having different physical properties in different directions. An anisotropic material reacts differently in different directions to the same applied stress. |
| Elastic | A behaviour range in which the applied stress is insufficient to result in permanent deformation when the stress is removed. |
| Plastic | A behaviour range in which the rock mass deforms at a constant applied stress. |
| Geological structure | Natural planes of weakness in the rock mass that pre-date mining. The term includes faults, folding, shears, joints, bedding places, foliation and schistosity. |
| Discontinuity | A mechanical break in the fabric of the rock mass across which there may or may not have been relative displacement. |
| Fracture | Any natural or mining-induced planar discontinuity between blocks of rock. Fractures may be tensile (extensional), in which case the fracture surfaces move further apart, or shear, in which case the fracture surfaces slide past one another. |
| Joint | A natural planar discontinuity between blocks of rock along which extremely little or no discernible (lateral) displacement has occurred. |
| Failure | A structure is considered to be in a failed state when it is no longer able to perform its intended task. In ground engineering, failure is usually caused by excessive movement along natural or mining-induced fractures. |
| Stable failure | Rock failure which occurs in a controlled manner; that is, with adequate warning and without the sudden release of large amounts of kinetic energy. |
| Unstable failure | Rock failure which occurs in an uncontrolled manner; that is, with little or no warning and with the sudden release of large amounts of energy |

very long in one direction, the final state of stress around it can be analysed in two dimensions provided that this stress field has not been seriously affected by deformation induced around the mining face during excavation. Although one-dimensional situations rarely exist in a three-dimensional mining environment, one-dimensional analysis is very useful for conceptualising and developing a basic understanding of the more important material properties that govern rock mass response and is applied extensively in this chapter.

### 2.5.1 Load-Displacement

When a material is subjected to compression or tension, it undergoes displacement. Conversely, compressing or stretching a material induces a change in load within the material. A system in which the load is increased to result in displacement is referred to as a **load controlled system**, while a system in which load is generated by increasing displacement is referred to as a **displacement controlled system**. Both these load-displacement circumstances arise in mining.

The simplest relationship between load, L, and displacement, Δd, is that shown in Fig. 2.6. This is for a material loaded in only one direction (uniaxial loading) and for which load is linearly proportional to displacement up to point A on the load-displacement curve. The material will return to its original dimensions if load or displacement are removed before point A is reached. Because the internal structure of the material progressively changes beyond this point, resulting in some permanent deformation of the material, Point A is referred to as the **yield point**.

Different loads are generated in different materials when subjected to the same displacement, and vice versa. **Stiffness**, k, is the engineering term used to describe the relationship between load and displacement and is given by

**Table 2.2** Definitions of some common physical parameters and their units of measurements

| Property | Symbol | Description | Unit of measurement and symbol |
|---|---|---|---|
| Distance | d | The spatial separation between two points of reference. | metre – m |
| Displacement | $\Delta d$ | Change in distance between two reference points. | m |
| Area | A | The number of square units of a certain size needed to cover the surface of a figure. | square metres – $m^2$ |
| Volume | V | The amount of space occupied by a material. | cubic metres – $m^3$ |
| | | In the case of a fluid, volume is expressed in terms of litres whereby 1 l is equal to the volume occupied by 1 kg of pure water at a temperature of 4 °C and a pressure of 760 mm of mercury. | litre – l |
| Time | t | A measure of the duration of events and the interval between them. | second – s |
| Mass | m | All bodies have a mass, which is the same wherever they are in the universe. | kilogram – kg |
| Velocity | v | Distance travelled (displacement) divided by time taken. That is, velocity is the rate of change of displacement. | m/s |
| Acceleration | a | Change in velocity over a given time. That is, acceleration is the rate of change of velocity. | $m/s^2$ |
| Force | F | When a mass undergoes acceleration, a force is generated. | $(kg) \times (m/s^2) = kg\ m/s^2$ |
| | | Force = Mass × Acceleration<br>$F = ma$ | |
| | | By definition, one kg $m/s^2$ equals one Newton | Newton – N |
| Gravitational acceleration | g | All bodies on the Earth are subjected to the Earth's gravity, that is, to gravitational acceleration, g. Typically, $g = 9.81\ m/s^2$ | $m/s^2$ |
| Weight | W | Gravity acts on all bodies on the Earth. Therefore, they are subjected to a force, which is usually referred to as weight. | $kg\ m/s^2 = N$ |
| | | Weight = Mass × Gravitational Acceleration<br>$W = mg$ | |
| Load | L | Another term for weight | N |
| Stress | $\sigma$ | The effect of a force on a body is relative to the area over which the force is distributed. Stress is the standard used to compare force (and weight or load) distributions. | $(kg) \times \left(\frac{m}{s^2}\right) \times \left(\frac{1}{m^2}\right) = \frac{kg}{m\ s^2} = kg/m\ s^2$ |
| | | Stress = Force per unit area<br>= Mass × Acceleration/Area<br>= F/A (or L/A or W/A) | |
| | | By definition, one $kg/ms^2$, or one $N/m^2$, equals one Pascal. | Pascal – Pa |
| Pressure | P | Another term for stress. Strictly speaking, the term applies to the hydrostatic state of stress that exists in fluids. | $kg/ms^2$ or Pa |
| Energy, or Work | | Is equal to force acting on a body x displacement caused by that force | N m = Joule – J |
| | | Work Done = Energy = Fd | |

(continued)

**Table 2.2** (continued)

| Property | Symbol | Description | Unit of measurement and symbol |
|---|---|---|---|
| Power | | The rate at which work is done | N m/s = Watt – W |
| | | Power = Fd/t | |
| Strain | $\varepsilon$ | The effect of displacement on the integrity of a material or structure is relative to the distance over which the displacement is distributed. Strain is the standard used to compare the deformation effects (lengthening or shortening) of displacement. | Dimensionless (m/m – Unitless) |
| | | Strain = Change in length/original length = $\Delta d/d$ | However, often expressed in terms of: mm/m |
| Longitudinal Strain | $\varepsilon_a$ | Often referred to as just ‘strain’, longitudinal (or axial) strain is a measure of the amount of deformation (change in length) parallel to a given line or axis. | Dimensionless |
| Lateral Strain | $\varepsilon_l$ | Is a measure of the amount of deformation (expansion or contraction) normal to the direction of an applied force. | Dimensionless |
| Volumetric Strain | $\varepsilon_v$ | Is the ratio of change in volume to the original volume. | Dimensionless |
| Density | $\rho$ | The mass of one cubic metre of a material. | $kg/m^3$ |
| | | $\rho$ = mass per unit volume | |
| Specific Weight | $\gamma$ | Weight per unit volume of a material of density, $\gamma = \rho g$ | $N/m^3$ |
| Normal Stress | $\sigma_n$ | The stress acting at right angles to a surface. | $kg/m\ s^2$ or Pa |
| | | $\sigma_n$ = force acting normal to a surface divided by the cross-sectional area of the surface. | |
| Shear Stress | $\tau$ | The stress acting along a plane that is sliding past another plane. | $kg/m\ s^2$ or Pa |
| | | $\tau$ = force acting parallel to a surface divided by the area of the surface over which the force acts. | |
| Shear Strain | $\varepsilon_s$ | Is a measure of the amount of deformation perpendicular to a given line as the internal structure of a body attempts to slide against itself. It is measured in terms of the tangent of the change in angle, $\alpha$, between two points of origin. | Dimensionless (Tan $\alpha$) |

Eq. 2.1. It is a measure of the ‘springiness’ of the material being loaded. In physics, the linear relationship between load and displacement up to point A is described by Hooke’s Law and stiffness is referred to as the **spring constant**.

$$\begin{aligned} Stiffness,\ k, &= \frac{Load}{Displacement} \\ &= \frac{L}{\Delta d} \quad \left(\frac{N}{m}\right) \end{aligned} \tag{2.1}$$

### 2.5.2 Stress-Strain

In order to evaluate the effects of load and displacement on a structure and to make comparisons, it is very useful to replot the load-displacement curve as a stress-strain curve, shown in Fig. 2.7, by normalising the load with respect to the area over which it acts and the displacement with respect to the distance, d, over which it occurs. This curve has the same

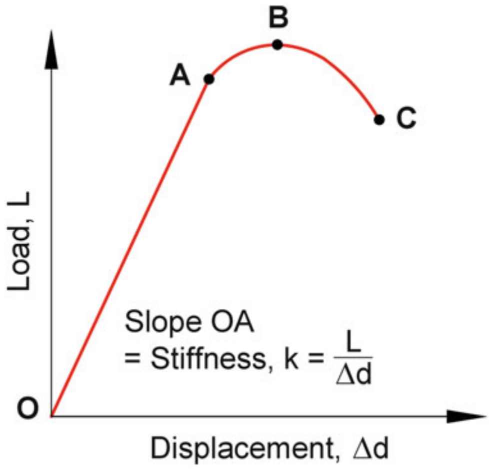


**Fig. 2.6** Load-displacement curve for a material with linear stiffness

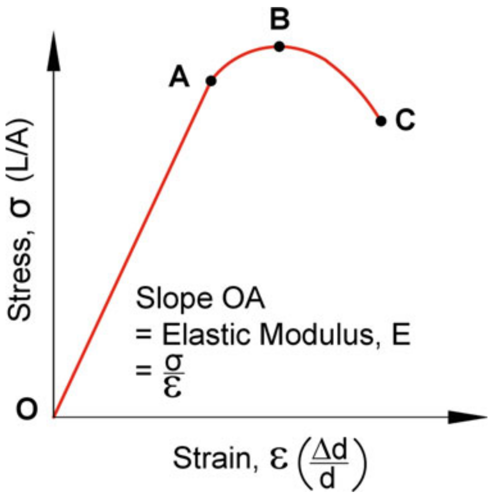


**Fig. 2.7** A stress-strain curve for a pure linear elastic material

form as the load-displacement curve, with the relationship between stress and strain up to the point A remaining linear and now being defined as the **Elastic Modulus** or **Young's Modulus**, E, as given by Eq. 2.2. Point A still defines the limit beyond which permanent deformation occurs and is known generally as the **elastic limit**. The term **limit of proportionality** defines the point beyond which the relationship between stress and strain is no longer linear.

$$\begin{aligned} \textit{Elastic Modulus, } E \quad &= \frac{\textit{Stress}}{\textit{Strain}} = \frac{\sigma}{\varepsilon} \\ &= \frac{Ld}{A\Delta d}\ (Pa) \end{aligned} \tag{2.2}$$

In practice, ground engineers work with many natural and man-made materials that do not exhibit linear behaviour up to the elastic limit and for which the elastic limit can be difficult to determine. The elastic modulus of these types of materials is expressed as either the **tangent modulus** or the **secant modulus**. The difference between these two moduli is illustrated in Fig. 2.8a. The tangent modulus is referenced to the level of stress at a point, P, which is often selected to be 50 % of the peak stress. The secant modulus is the slope of the line drawn through this point and the origin, O. In the case of fissured materials, a degree of displacement may be required to close up pore spaces in order to generate load, effectively moving the origin to point O′ as in Fig. 2.8b. Some typical values for Young's Modulus are presented in Table 2.3. Note that even the strongest rocks do not have a high modulus when compared to many manufactured metal and ceramic construction materials.

### 2.5.3 Stiffness

By combining Eqs. 2.1 and 2.2, stiffness can be expressed as:

$$\textit{Stiffness, } k, \ = \frac{L}{\Delta d} = \frac{\sigma A}{\varepsilon d} = \frac{EA}{d} \qquad \left(\frac{N}{m}\right) \tag{2.3}$$

Hence, load generated by displacement is given by:

$$\textit{Load, } L, \ = \frac{EA\Delta d}{d} \quad (N) \tag{2.4}$$

Equations 2.3 and 2.4 constitute two of the most important foundations for understanding ground response to mining. It is important to appreciate that:

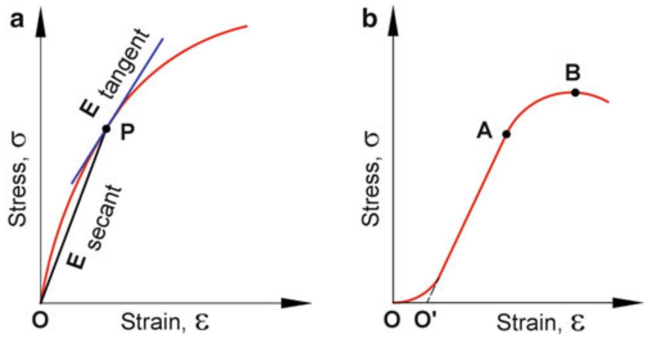


**Fig. 2.8** Characteristic stress-strain curves for non-linear elastic materials and for fissured materials (**a**) A non-linear elastic stress-strain curve (**b**) A stress-strain curve for a fissured rock

**Table 2.3** Typical values quoted in the literature for Young's Modulus

| Material | Typical Young's Modulus (GPa) |
|---|---|
| Coal | 2–4 |
| Sandstone | 6–30 |
| Conglomerate | 20–40 |
| Granite | 15–75 |
| Concrete | 30–50 |
| Dolerite | 40–80 |
| Aluminium | 69 |
| Glass | 50–100 |
| Steel | 200–210 |
| Tungsten | 400–410 |
| Silicon carbide | 400–450 |

- Stiffness is not the same as elastic modulus.
- Elastic modulus is a material property.
- Stiffness is a function of both a material property (elastic modulus) and geometry (area and height or length) and, therefore, is a structural property.
- A body subjected to load can be conceptualised as a spring up to the onset of yield.

### 2.5.4 Strength

**Strength** is defined as the maximum or peak stress that a structure can sustain, corresponding to point B in Fig. 2.7 (and in Fig. 2.6). In a displacement controlled system, strength equates to a structure's peak resistance to deformation. Because stress can vary throughout a structure, strength is often expressed in terms of **peak average stress**, calculated by dividing the total load acting on the structure by the area over which the load acts.

At some point between the elastic limit and the peak stress, the material is considered to have reached its **yield strength**. Two definitions of yield strength find application in ground engineering, these being '*the stress at which the material begins to deform plastically*' and '*the stress at which a predetermined amount of permanent deformation occurs*'. The first definition is applied extensively to rock and the second to ground support elements fabricated from metal. The concept of **proof load**, being the load required to produce a specified total strain (elastic and inelastic), is sometimes applied instead of yield strength. Cable bolt performance, for example, is often specified on the basis of proof load.

### 2.5.5 Stored Energy and Seismicity

The application of force to a surface results in displacement, and vice versa. This cause and effect relationship defines **work done**, given by Eq. 2.5.

$$\mathit{Work\ Done} = \mathit{Force} \times \mathit{Displacement} \tag{2.5}$$

Work done is a measure of the energy required to cause displacement or to generate a force. In theory, up until a material begins to yield, all the energy required to generate displacement or force is stored as **strain energy**. This energy corresponds to the **potential energy** contained within a compressed spring and is equal to the area under the load-displacement curve. Once a body is loaded beyond its yield point, a greater proportion of the additional energy input is consumed in permanently distorting the structure of the body, in the expulsion of failed elements of the body and by processes such as the generation of noise and heat.

The potential energy, U, stored in a volume of material, V, that has been subjected to a linear elastic displacement, Δd, due to a force or load, L, (or vice versa) is given by Eq. 2.6.

$$\begin{aligned} \textit{Potential Energy}, U &= \frac{1}{2} L\Delta d = \frac{1}{2}\sigma\varepsilon A d = \frac{1}{2}\sigma\varepsilon V \\ &= \frac{1}{2}\frac{\sigma^2}{E} V \quad (J) \end{aligned} \tag{2.6}$$

Hence, **strain energy per unit volume**, $W_e$, up to the point of yield is given by Eq. 2.7.

$$W_e = \frac{1}{2}\sigma\varepsilon = \frac{1}{2}\frac{\sigma^2}{E} \quad \left(\frac{J}{m^3}\right) \tag{2.7}$$

Rock fracturing results in the release of energy in the form of elastic, or seismic, waves which radiate out from the locus of the fracture site. It is the sudden conversion of a large amount of stored energy into kinetic energy that accounts for dynamic events that are variously referred to in coal mining as **pressure bursts**, **strain bursts**, **rockbursts** and **bumps.** These are discussed in more detail in Sect. 11.8. A seismic monitoring network, comprising an array of vibration sensors, which are usually geophones, can be used to determine fracture type (intact shear, intact tensile, bedding plane shear, reactivation of a shear plane etc), location and magnitude by detecting and measuring arrival times, amplitudes and duration of ground vibrations.

The phenomenon of acoustic emission is attributed to the growth of fractures. Wagner (1974) and Hatherly et al. (2003) report that field measurements indicate that microfracturing is initiated at stress levels of about half the rock strength. An increase in the rate of microseismic emissions is a precursor to rock failure, indicating an increasing rate of crack growth and accelerating failure (Hatherly and Luo 1999). By accumulating the foci of rock failures over a period of time, it is possible to delineate the orientation and extent of a fracture zone.

### 2.5.6 Poisson's Effect

When a compressive stress is applied to a body, it is pushed and will shorten. This generates compressive strain in the direction of the applied stress, referred to as the axial direction. At the same time, the body will expand laterally and generate tensile strain in the orthogonal direction, as shown in Fig. 2.9. Conversely, a tensile stress will pull the body and stretch it in the direction of the applied stress, causing it to thin laterally.

The ratio between the **axial strain** and the **lateral strain** induced in a body by a force is known as **Poisson's ratio**, ν, and is defined by Eq. 2.8.

$$\textit{Poisson's ratio}, \nu = \frac{-\varepsilon_l}{\varepsilon_a} = -\frac{\varepsilon_l}{\varepsilon_a} \tag{2.8}$$

Figure 2.9 illustrates that at any given point in the loading cycle, lateral strain is considerably higher than axial strain. This is because compression gives rise to the important phenomenon of **dilation**, which is associated with the opening of internal fractures that are aligned in the direction of maximum applied compressive stress. This phenomenon distinguishes rock behaviour from that of most other materials.

The Poisson's ratio of most engineering materials ranges from 0 to 0.5, with 0.5 defining a perfectly incompressible material. The value for rubber is close to 0.5 while that for cork is close to 0. Some materials become thicker in the direction perpendicular to the applied force when they are stretched and, therefore, have a negative Poisson's ratio. These are referred to as **auxetic**. Typical values for Poisson's ratio are presented in Table 2.4.

**Fig. 2.9** Behaviour of a prism under load (Adapted from Cook et al. 1974)

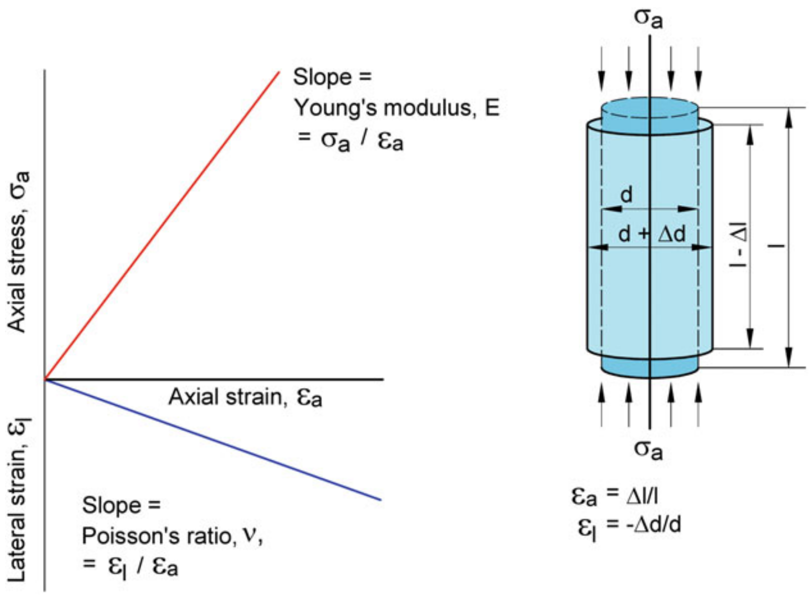


**Table 2.4** Typical values quoted in the literature for Poisson's ratio

| Material | Poisson's ratio |
|---|---|
| Cork | ~0 |
| Concrete | 0.1–0.2 |
| Sedimentary rock | 0.2–0.25 |
| Cast iron | 0.21–0.26 |
| Glass | 0.22–0.24 |
| Stainless steel | 0.30–0.31 |
| Copper | 0.33–0.36 |
| Rubber | 0.48 – ~0.5 |

### 2.5.7 Cohesion and Friction on a Fracture Surface

**Cohesion**, c, is a measure of the strength of the intermolecular forces within a material. It corresponds to the shear load required to cause sliding between two surfaces that are not acted upon by normal stress. Hoek et al. (1995) explain that cohesion is a soil mechanics term that has been adopted by the rock mechanics community. There is a distinction between the two applications that needs to be appreciated. In shear tests on soils, the stress levels are generally an order of magnitude lower than those involved in rock testing and the cohesive strength of a soil is a result of the adhesion of the soil particles. In rock mechanics, cohesion is a measure of the bond strength of 'cemented' surfaces. Nevertheless, in most practical rock mechanics applications, the term 'cohesion' is used for convenience.

**Friction** is a measure of the natural resistance to relative lateral motion, or sliding. In the case of an existing fracture plane, it corresponds to the angle of tilt required to cause one surface to slide past the other in the absence of cohesion. This tilt angle is referred to as the **angle of friction** or the **friction angle,** $\phi_s$, with $\tan(\phi_s)$ known as the **coefficient of static friction**, $\mu_s$, defined in Eq. 2.9. The **coefficient of friction** corresponds to the ratio between the **shear stress**, $\tau$, required to cause a body to start to slide on a cohesionless surface and the **normal stress**, $\sigma_n$, acting across the surface. The angle of static friction is comprised of two components, the first being a friction angle, $\phi_b$, associated with a smooth fracture surface and the second being a friction angle, i, that accounts for asperities on the fracture surface (see Sect. 2.6.5). Consequently, the angle of friction is subject to considerable variability, depending on factors such as surface composition, cleanliness, lubrication and finish. The symbol for static angle of friction is often abbreviated to simply, $\phi$.

**Table 2.5** Typical values quoted in the literature for friction angle and coefficient of friction

| | Angle of friction, ϕ, and coefficient of friction, μ | | | |
|---|---|---|---|---|
| | Static | | Dynamic | |
| Contacting surfaces | $\phi_s$ | $\mu_s$ | $\phi_d$ | $\mu_d$ |
| Teflon on steel | 2.3° | 0.04 | 2.3° | 0.04 |
| Steel on steel (dry) | 31°–38.7° | 0.6–0.8 | 22° | 0.4 |
| Steel on steel (greasy) | 2.9–11.3° | 0.05–0.2 | 1.7°–8.5° | 0.03–0.15 |
| Wood on wood (dry) | 14°–26.6° | 0.25–0.5 | – | – |
| Wood on wood (wet) | 11.3° | 0.2 | – | – |
| Clay, shale, talcose shear planes | 11.3° | 0.2 | – | – |
| Rubber tyres on dry pavement | 42° | 0.9 | 38.7° | 0.8 |
| Ice on ice | 5.7° | 0.1 | 1.7° | 0.03 |

Note: These values are indicative only and are subject to considerable variability

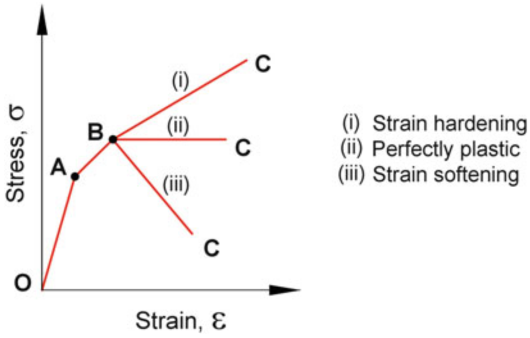


**Fig. 2.10** Post-peak strength behaviour modes of materials

$$\mu_s = tan\,(\phi_s) = tan\,(\phi_b + i) \qquad (2.9)$$

A material that is so highly jointed or shattered that there is zero cohesion between particles will form a pile that has a slope, or **angle of repose,** equal to or less than the angle of static friction. The coefficient of friction between two surfaces that are already sliding past each other is referred to as the **coefficient of kinetic friction** or **coefficient of dynamic friction**, which is usually less than the coefficient of static friction. Some typical static and dynamic friction values are given in Table 2.5.

### 2.5.8 Post-peak Strength Behaviour

Once its peak strength has been exceeded, a material behaves in one of the three general manners shown in Fig. 2.10. Curve (i) illustrates **strain hardening**, in which the load carrying capacity of the material continues to increase with further displacement but at a reduced rate to that before the onset of yield. Curve (ii) illustrates **perfectly plastic** behaviour, in which the material will maintain load with ongoing displacement or, conversely, continue to displace at a constant load. These two types of behaviour are also referred to as **ductile behaviour** and are characteristic of metals. Curve (iii) illustrates **brittle behaviour**, which is characteristic of rocks, glass and cast iron. Once the peak load carrying capacity has been reached, these types of materials shed load with further displacement. This behaviour is referred to as **strain softening**.

The slope of the post-peak strength section of the stress-strain curves, B-C, in Fig. 2.10 is referred to as the **post-peak modulus**. Unlike the **pre-peak modulus**, or elastic modulus, the post-peak modulus is not a material property but a function of the geometry (width, height and length) of the material. The shape of the post-peak section of the corresponding load-displacement curve mirrors that of the stress-strain curve and defines the **post-peak stiffness**.

## 2.6 Rock Mechanics

### 2.6.1 Specifying Stresses within Rock

Yielding and load shedding in a rock mass arise from either or both the formation of one or more

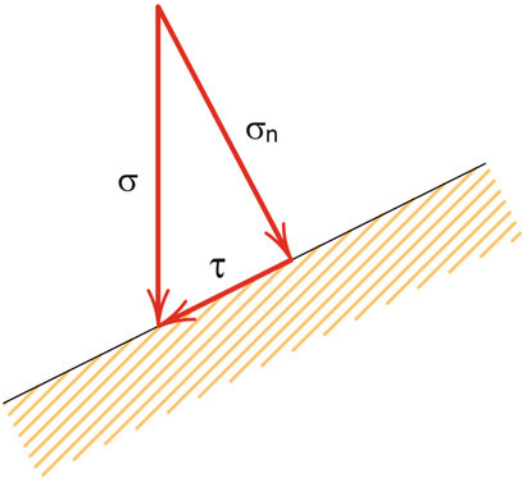


**Fig. 2.11** Resolution of a stress, σ, acting on a surface into its normal, $\sigma_n$, and shear, τ, components

new fracture surfaces within the rock mass and from sliding on existing fracture surfaces. Unlike fluids, solids have the capacity to sustain shear stresses, within limits. Therefore, any stress, σ, acting on a fracture surface or plane can be resolved into a 'normal' component, $\sigma_n$, at right angles to the plane and a 'shear' component, τ, parallel to the plane (Fig. 2.11). A stress which acts normal to a surface does not induce shear and is referred to as a **principal stress**. The plane on which this stress acts is called a **principal stress plane** and is free of shear stresses.

The state of stress within a body can be defined by three orthogonal principal stresses. The **maximum principal stress** (or **major principal stress**) is designated $\sigma_1$, the **intermediate principal stress** is $\sigma_2$, and the **minimum** principal stress (or **minor principal stress**) is $\sigma_3$. The difference between the major and minor principal stresses, $\sigma_1\_\sigma_3$, is referred to as the **deviator stress**.

In order to undertake analysis in three dimensions, stress directions are defined in terms of either a polar coordinate system, based on radius, r, and a subtended angle, θ, or a Cartesian coordinate system based on x, y and z axes. In the most general case, stresses are denoted by the symbol $\sigma$ (sigma), together with subscripts to define the surface on which the stress acts and the direction of the stress The convention in the Cartesian coordinate system, used in this text, is that the first subscript indicates the direction of a normal to the surface on which the stress acts and the second subscript indicates the direction of the stress acting on the surface. For example, in Fig. 2.12, $\sigma_{xx}$ defines a normal stress acting on the yz plane and orientated parallel to the x axis, and $\sigma_{xz}$ defines a shear stress acting on the yz plane orientated parallel to the z axis. By definition, $\sigma_{xx}$, $\sigma_{yy}$, and $\sigma_{zz}$ are principal stresses and their designation is often abbreviated to $\sigma_x$, $\sigma_y$, and $\sigma_z$. The symbol $\tau$ (tau) can be used in place of σ to denote shear stress.

Similarly, the symbol ε (epsilon) is used to denote **strain**, with γ (gamma) sometimes being used to specifically denote **shear strain**. The same convention as for stress applies to the use of subscripts to denote strain directions and affected surfaces.

Any state of strain in a body can be specified by three orthogonal normal strains, defined as **principal strains**. A normal strain in a particular direction does not necessarily imply that a normal stress is also acting in that direction. Rather, the horizontal strain may be due to unrestricted expansion of the body in that direction under the effect of a normal stress acting in another direction.

In the case of an underground excavation, stress acting around the perimeter of the excavation can be resolved into a normal, or radial component, and a tangential component. The **radial stress** acting on the walls corresponds to barometric pressure and, therefore, is considered to be zero for all practical purposes. The **tangential stress** varies around the perimeter and, depending on the external stress field, may be tensile in some sections and compressive in others.

A range of analytical solutions exist for calculating elastic stress distributions around isolated excavations located in a homogenous medium, an example of which is shown in Fig. 2.13. These can be very useful for conceptualising where and how rock mass fracturing might develop. However, they have limited direct application in many underground coal mining environments because of the influence of

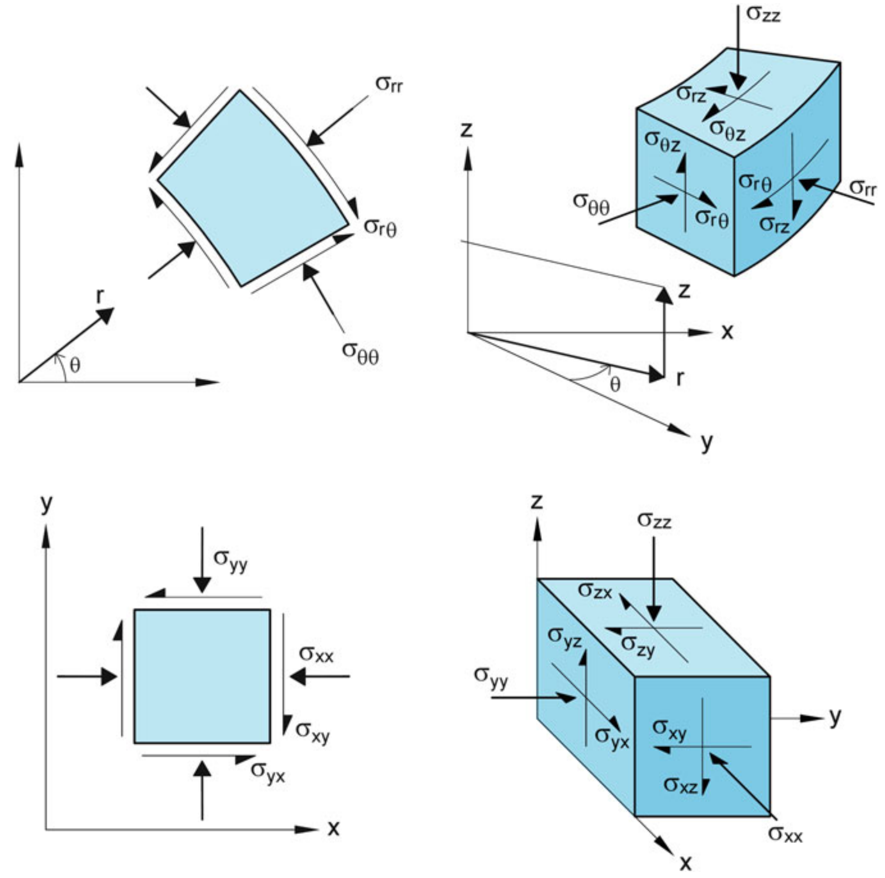


**Fig. 2.12** Illustration of the geomechanics convention for naming stresses (Adapted from Brady and Brown 2006)

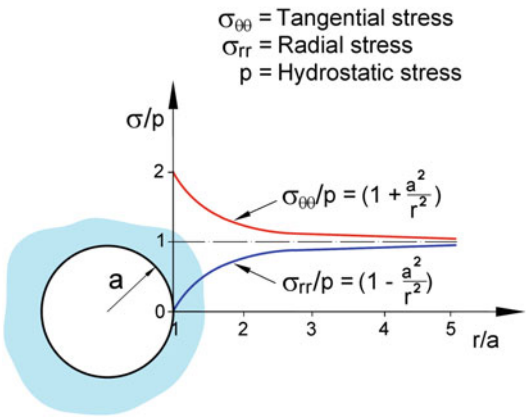


**Fig. 2.13** Axisymmetric stress distribution around a circular opening in a hydrostatic stress field determined with the aid of an analytical solution (After Brady and Brown 1993)

bedding on rock mass response; the complex and often unknown nature of the regional stress field at any point in time; the inability to analyse stress in a three-dimensional setting about an active coal face; and the inability to evaluate how deformation is influenced by stress path history.

The end-user needs to be alert to a number of peculiarities associated with stress analysis conventions in ground engineering. Firstly, there is no fixed convention for designating in which direction θ is measured when using polar coordinates. Hoek and Brown (1980) for example, measure θ in an anticlockwise direction while Brady and Brown (2006) measure it in a clockwise direction. Stress and strain formulations need to be adjusted accordingly. Secondly, the convention for the direction of

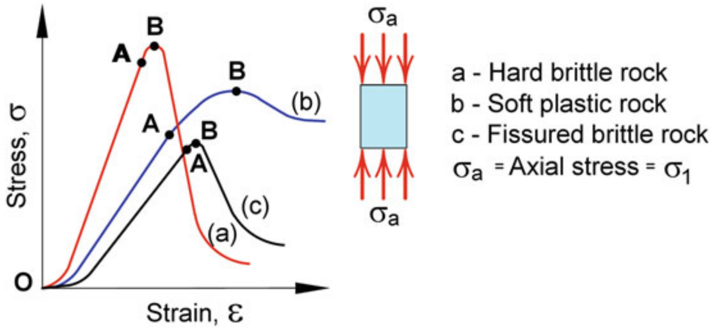


**Fig. 2.14** Deformation characteristics of different rock types when loaded in uniaxial compression in a stiff testing machine (Adapted from Salamon and Oravecz 1976)

forces is the opposite to that adopted in elastic theory, engineering mechanics and many numerical modelling codes, where stresses and strains are usually reckoned to be positive when tensile. As explained by Jaeger and Cook (1979), it is more convenient in rock mechanics to have compressive stresses and strains positive for the following reasons:

- Environmental stresses, such as overburden stress and fluid pressure are always compressive.
- The convention is more consistent with that adopted in the associated disciplines of soil mechanics and structural geology.
- Many problems in rock mechanics involve friction over surfaces, and in this case the normal stress is necessarily compressive.

This change in sign convention leaves all stress and strain formulae unaltered but can lead to erroneous results if not appreciated when working with some numerical modelling codes.

### 2.6.2 Strength of Rock

The deformation behaviour of rock up to its yield point is generally similar to that shown in the idealised curve in Fig. 2.8b. Thereafter, different rock types deform in different manners, as illustrated in Fig. 2.14 for the simplest case of uniaxial compression. Curve ‘a’ is typical of the behaviour of hard brittle rocks, such as conglomerates and dolerites and curve ‘b’ of soft plastic rocks, such as shales. The behaviour of fissured brittle rocks, such as coal, is characterised by curve ‘c’.

The distance between points A and B in brittle rocks (curves ‘a’ and ‘c’) is quite small. However, once plastic rocks reach yield (curve ‘b’), they can still undergo considerable displacement (‘flow’) in a stiff loading environment (see Sect. 2.6.11) before reaching their peak load carrying capacity. Two important points to note are:

- Rock is still capable of carrying a considerable load in an overloaded or failed state.
- Rock can continue to deform for long periods of time when subjected to constant stress. This time-dependent strain behaviour, depicted in Fig. 2.15, is known as **creep**. It is small in the case of hard rocks such as dolerites and conglomerates but can be appreciable in shales, mudstones and claystones, especially in the presence of water. The higher the stress that the rock is subjected to, the more likely it is to deform over time. Consequently, the strength of many rock types can decrease over time. This is particularly the case in coal mining environments because many sedimentary rock types are relatively soft and weak and some, including coal, are aquifers and therefore a source of water.

One of the conundrums in rock mechanics is that rock cannot be assigned a unique value of strength. This is because strength is a function of many parameters that include:

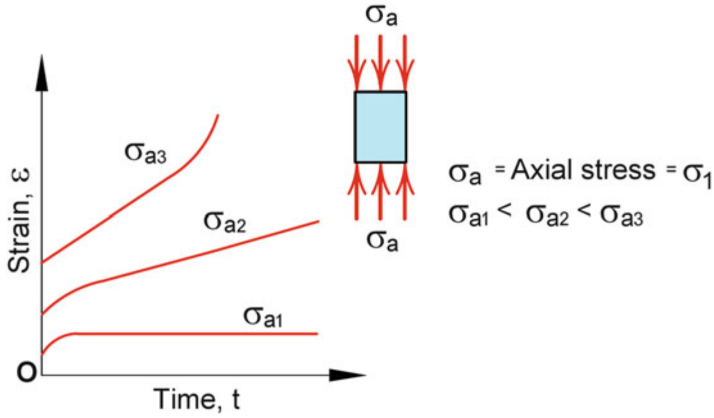


**Fig. 2.15** Creep behaviour of rocks under different levels of constant stress (Adapted from Salamon and Oravecz 1976)

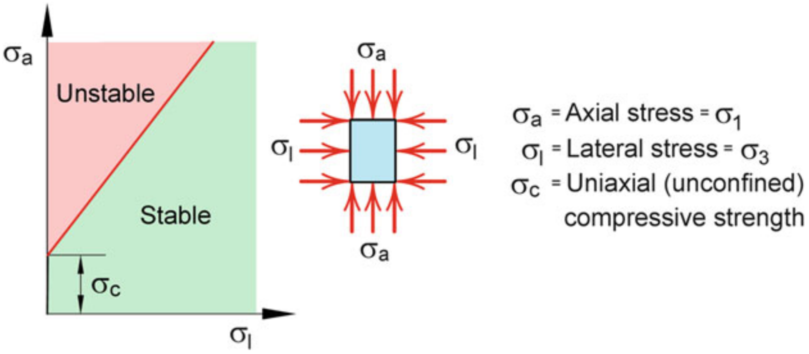

$\sigma_a$
Unstable
Stable
$\sigma_c$
$\sigma_l$
$\sigma_a$
$\sigma_l$
$\sigma_l$
$\sigma_a$
$\sigma_a$ = Axial stress = $\sigma_1$
$\sigma_l$ = Lateral stress = $\sigma_3$
$\sigma_c$ = Uniaxial (unconfined) compressive strength

**Fig. 2.16** The influence of confining stress on the strength of rock (Adapted from Salamon and Oravecz 1976)

- Lateral confinement. The compressive strength of a brittle material like rock increases with increase in lateral confinement as illustrated in Fig. 2.16. The strength of rock when it is not subjected to lateral confinement is referred to as the **uniaxial compressive strength** (UCS), or **unconfined compressive strength**, $\sigma_c$. The strength of rock subjected to lateral confinement is referred to as the **triaxial compressive strength**, $\sigma_1$, and is specific to the applied confining pressure.
- The type of applied stress. Typically, rocks are approximately 10 times weaker in tension than in compression but the difference may exceed 30-fold.
- The direction of the applied stress. Strength is not the same in all directions in rocks which are not isotropic.
- Size. As size increases, the strength of most rock types reduces to a lower limit due to the impact of natural defects in the rock mass. Some relationships determined from compressive testing are shown in Fig. 2.17.
- Time. When subjected to sustained elevated stress, the strength of some rocks reduces over time.

Another means of illustrating the effect of lateral confinement on rock strength is to plot deviator stress against **deviator strain**, being the difference between axial and lateral strain (Fig. 2.18). In addition to illustrating how strength increases with increase in confinement, Fig. 2.18 also illustrates another very important principle; that is, the **residual strength** of failed rock also increases with increase in confinement.

Given the beneficial effects of confinement on rock strength, it is only to be expected that deformation of the rock mass will be most severe around the unconfined surfaces of an excavation.

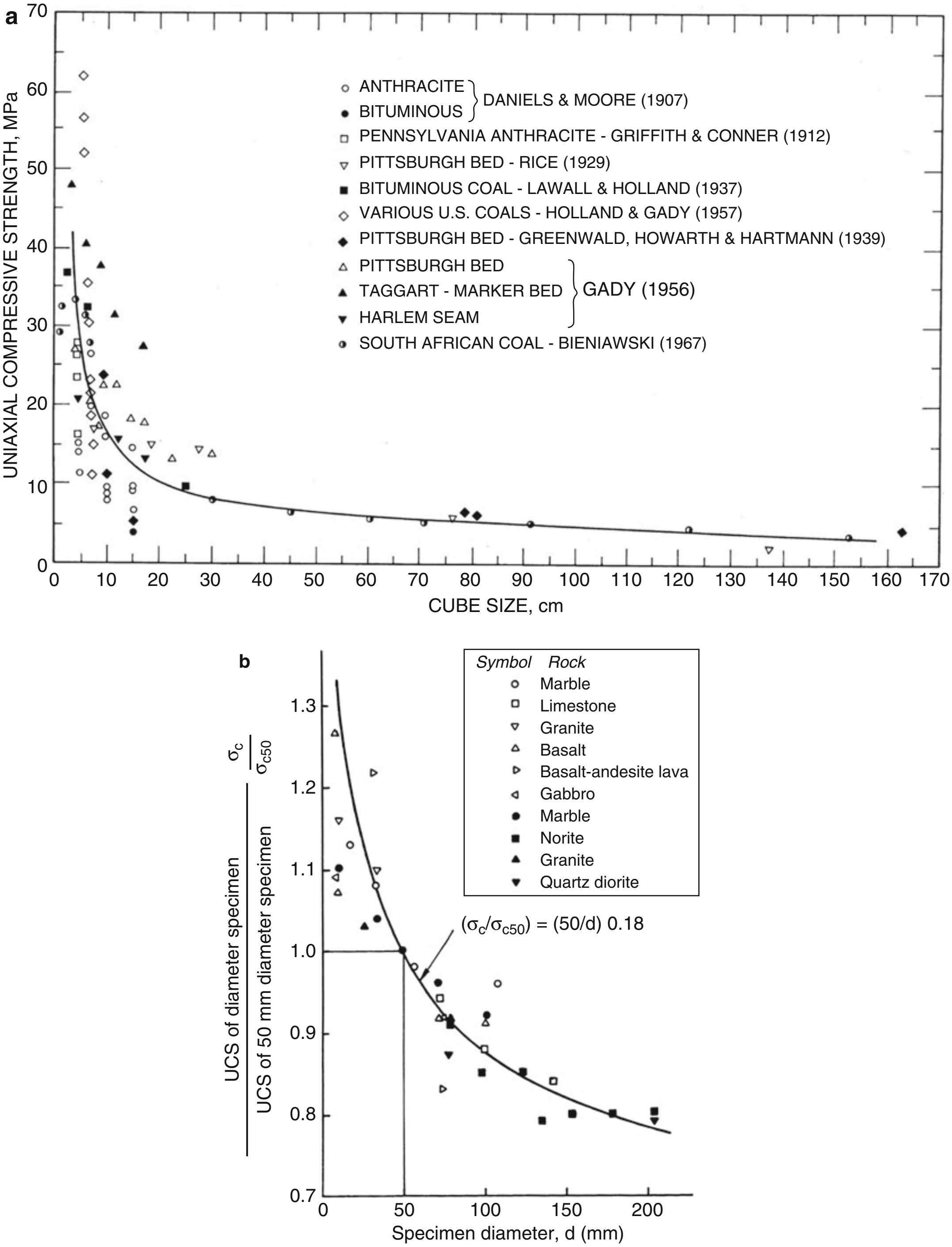


**Fig. 2.17** Examples of the effect of size on the compressive strength of coal and hard rock specimens (**a**) Coal cubes (After Singh 1981, Chapter 5: Strength of rock. In physical properties of rock and minerals, Vol. II-2. Reproduced with permission of McGraw Hill Education) (**b**) Hard rock types (Adaptation from Hoek and Brown 1980, courtesy of Professor Hoek)

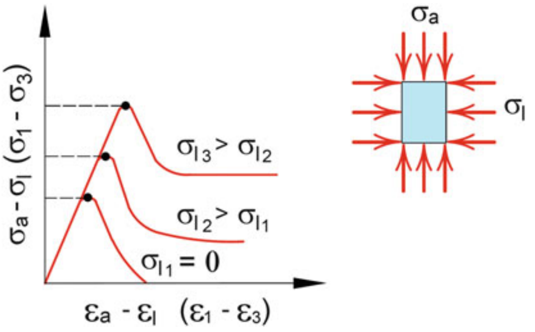


**Fig. 2.18** The influence of confinement on the strength and post-peak behaviour of rock

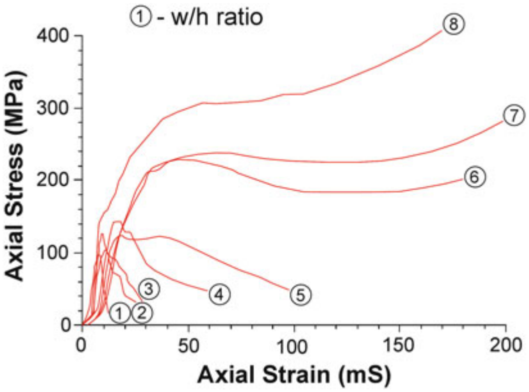


**Fig. 2.19** The influence of width-to-height ratio on the strength and post-peak behaviour of 24 mm diameter sandstone specimens (After Madden 1990)

As deformation progresses into the rock mass, a degree of self confinement is generated by the outer material even though it is in a post-peak state, thereby increasing rock strength. The benefits of self confinement can be observed in the laboratory by increasing the width-to-height ratio, w/h, of specimens subjected to axial compression, as illustrated in Fig. 2.19. The laboratory results also illustrate how the behaviour of rock progressively changes from a brittle mode to a ductile mode as confining stress is increased. However, the strength values and stress-strain behaviours plotted in Fig. 2.19 should not be taken to be truly representative of in situ strength and behaviour because they have been influenced by stress fields specific to the end conditions that are not encountered in situ.

The preceding characteristics of rock have a number of implications for ground engineering, especially in a mining environment. In particular:

- Laboratory strength values are specific to the shape, width-to-height ratio and volume of the test specimens and are very unlikely to be representative of rock mass strength in the field.
- The strength of rock surrounding a roadway varies with distance into the rock mass.
- Secondary mining operations may change not only the load acting on the surrounding rock mass but also the strength of this rock mass because of mining-induced changes in rock mass confinement.
- The analysis of the state of stability of a mining system at any point in time must consider the stress regimes that the system has already been subjected to over its lifetime. This history is referred to as the **stress path**.

The determination of rock mass strength from intact rock strength is a particularly vexing issue. In the first instance, it may not be possible to source or prepare samples for testing from weaker horizons. Hence, less is likely to be known about those materials which have the greatest impact on rock mass response. The sample collection, transport and preparation process can constitute a testing program in its own right, with survival being biased towards stronger samples. Numerous factors can then affect the outcomes of laboratory testing, including specimen volume, shape, moisture content, preparation standard, surface properties of the testing machine platens, loading system and loading rate.

The influence of testing machine platens on the laboratory strength of rock was studied by Brown and Gonano (1974). The authors conducted two suites of compressive testing on 51 mm diameter samples of Wombeyan Marble, one suite loaded between solid steel platens and the other between brush platens constructed from an assembly of 3.2 mm square high tensile pins.

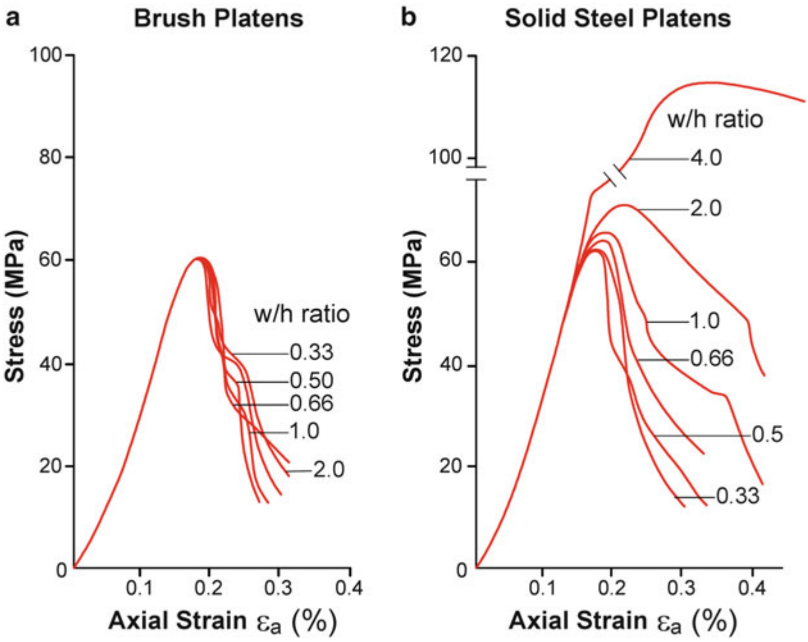


**Fig. 2.20** Influence of end constraint on the stress-strain behaviour of 51 mm specimens of Wombeyan Marble over a width-to-height range of 0.33–4 (Adapted from Brown and Gonano 1974)

The significant difference in stress-strain behaviour is illustrated in Fig. 2.20. Hustrulid and Ramos (1978) undertook uniaxial compressive strength testing using a lower platen sub-divided into a matrix of individual platens, each with its own load cell. They concluded that the post-failure curves were similar to those reported by Cook et al. (1971) and Wagner (1974) for in situ coal pillar tests and quite different from the triaxial results obtained by Crouch and Fairhurst (1973) when testing the same coal.

The International Society of Rock Mechanics (ISRM) has developed standards for laboratory testing which address factors such as sample dimensions and specimen preparation. Nevertheless, variable laboratory testing outcomes are still to be expected because rock is a natural material that contains inherent variations and defects in its fabric. McNally (1996), for example, reported that the standard deviation for UCS testing is typically 35–45 % of the mean.

Because laboratory determined rock strengths are relative rather than absolute, the shape, width-to-height ratio and volume of the test specimens need to be taken into account when working with rock strength values. Notwithstanding this, even if absolute values for strength and other material properties were to be obtained from laboratory testing, questions would remain as to how representative these are of the properties of the rock mass.

A number of researchers have attempted to test large scale samples to provide insight into the effects of scale on the compressive strength of rock. Bieniawski (1969), for example, employed a combination of laboratory and purpose-built apparatus to test a suite of cubical samples underground. The results indicated that the strength of a 25 mm cube of coal was of the order of seven times that of a 1.5 m cube of coal. Medhurst and Brown (1996, 1998) utilised 61, 101, 146 and 300 mm diameter cylindrical specimens with a width-to-height ratio of 0.5, such that the mid-height of the specimens fell outside of the influence of the platen interfaces, to study the effects of scale on the mechanical strength of coal. Their test results indicated that the average peak strength of coal decreased with increasing sample size. The researchers also applied the Hoek-Brown criterion (see Sect. 2.6.4) to estimate coal mass strength, concluding that it was approximately 25 % below the average value of commonly employed cube strength values. They noted that this agreed well with the results of Townsend et al. (1977) who found that small cylindrical samples were typically 20–30 % weaker than cubical samples of the same cross-sectional area.

The strength of harder rock types is also a function of size (or volume) as shown in Fig. 2.17b. Hoek and Brown (1980) defined this graphical relationship by Eq. 2.10.

$$\sigma_c = \sigma_{c50}\left(\frac{50}{d}\right)^{0.18} \tag{2.10}$$

where

$\sigma_{c50} = UCS\ of\ a\ 50mm\ diameter\ specimen$
$d = diameter\ of\ specimen\ in\ mm$

Although the relationship described by Eq. 2.10 is based on cylindrical samples of 'hard' rock up to only 200 mm in diameter, it is often invoked to estimate strength of not only 'soft' rocks but also of rock blocks with widths of up to 2 m or more. It is not uncommon, for example, to find in situ rock being equated to a specimen having a diameter of 1 m, with substitution into Eq. 2.10 yielding an in situ strength of 0.58 of that determined from the laboratory testing of 50 mm diameter specimens. This compares to a predicted in situ strength of only about 0.15 times the laboratory determined strength for coal based on Fig. 2.17a.

The contrast in estimates of in situ strength is one illustration of how empirical techniques based on laboratory testing and extrapolation of outcomes are fraught with uncertainty. Such approaches need to be underpinned, firstly, by a sound understanding of the limitations of laboratory testing, particularly in respect to the testing of cubic and rectangular shaped specimens; secondly, by field experience; and thirdly, preferably by stochastic analysis. For example, while Fig. 2.17a is useful in demonstrating scale effects and is referenced extensively in this regard, it is not a true measure of uniaxial compressive strength. This is because the width-to-height ratio of a cube, (being one), is not small enough to place the mid-height of a cubic specimen beyond the influence of the shear stress fields generated at the specimen/loading platen interfaces.

### 2.6.3 Equivalent Modulus of Strata

Almost invariably, coal mining takes place in environments in which the surrounding rock mass is comprised of a variety of strata of differing thickness that are transversely isotropic in behaviour. Convergence of the superincumbent strata is determined by the overall stiffness of the strata, which is a function of excavation span, excavation depth and the equivalent elastic modulus of the strata normal to the bedding, $E_{eq\text{-}n}$. Situations also arise where the behaviour of interest is governed by the equivalent elastic modulus parallel to bedding, $E_{eq\text{-}p}$. Solutions derived by Salamon (1968) for the equivalent modulus of transversely isotropic strata for both these situations are presented in Appendix 2.

The practitioner needs to be alert to some ground engineering designs that are based on equivalent moduli derived on the basis of force-displacement behaviour in only one-dimension. Examples are to be found in pillar design in layered floor strata settings and the assessment of the performance of backfill that has been placed in layers. Whilst these formulations are quick and convenient to use and can give reasonable predictions of behaviour in many cases, they are not mechanistically rigorous in most ground engineering applications and should not be relied upon in critical situations.

### 2.6.4 Failure Criteria

The relationship between stress and strain in a material is referred to as **constitutive behaviour**. A variety of idealised constitutive models, or **constitutive laws**, have been formulated to describe this behaviour. Elastic theory forms the

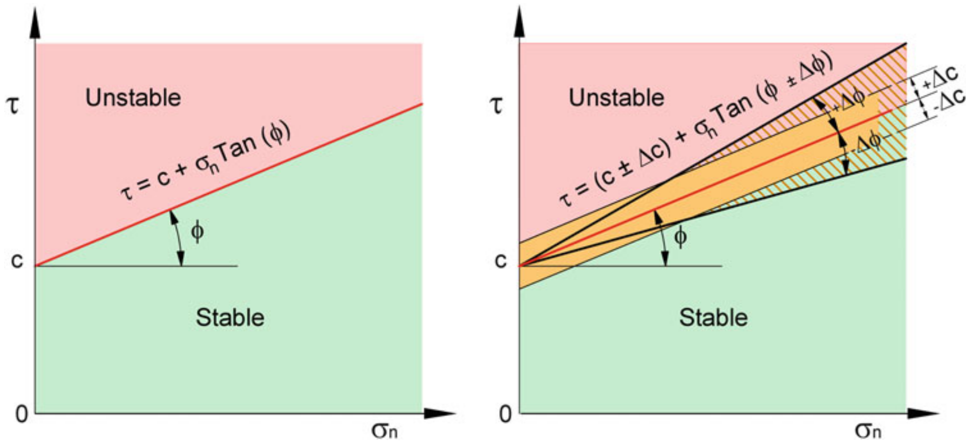


**Fig. 2.21** Coulomb failure criterion and the effect of inaccuracies in estimates of cohesion and friction

basis of nearly all the constitutive models up to the point of peak stress. Post-peak behaviour is most often described by the models in terms of brittle behaviour, plasticity, viscosity and creep. The deformation process in rock is complex in its own right and complicated further by influencing factors such as the effect of size, shape, loading rate and time on rock strength. Not surprisingly, therefore, there is no one constitutive law or failure criterion that is mechanistically rigorous or universally applicable.

Failure criteria are premised on threshold levels of either stress, strain, or energy. Three stress based failure criteria that have found extensive application in rock mechanics are those of Coulomb, Mohr, and Hoek and Brown. The Coulomb criterion is the oldest, dating back to 1773, and is based on the mechanical testing of rock. Coulomb suggested that shear failure, or sliding, occurs when the shear stress, τ, acting along a potential plane of weakness exceeds the sum of the cohesion, c, and the frictional resistance, $\sigma_n$.tanϕ, on that plane, as defined by Eqs. 2.11 and 2.12. Note that the **angle of internal friction** is not the same as the **angle of friction** discussed in Sect. 2.5.7. The former is associated with overcoming internal resistance to the formation of a new fracture in intact material, while the latter is a measure of friction on the surface of an existing fracture.

$$\tau \geq c + \sigma_n tan\,\phi \tag{2.11}$$

*or*

$$\tau \geq c + \mu\sigma_n \tag{2.12}$$

where

$c = cohesion$
$\phi = internal\,angle\,of\,friction$

The linear relationship between normal and shear stress proposed by Coulomb is shown in Fig. 2.21a, with the effect on shear strength of varying the values of cohesion and friction shown in Fig. 2.21b. In a low stress environment, variations in cohesion have the greater impact on shear strength. Variations in the friction angle have an increasingly greater impact on shear strength as normal stress increases. Hence, in low stress environments the focus needs to be more on accurately determining cohesion, while in high stress environments it is more critical to accurately determine the coefficient of friction.

In the early 1880s, Mohr hypothesised that the normal stress, $\sigma_n$, and the shear stress, τ, across a plane at the onset of shear failure are related by a function that is a characteristic of the material and not necessarily linear. This relationship is usually derived from laboratory testing to determine a range of maximum and minimum principal stresses at the point of shear failure. These are plotted in the manner shown in Fig. 2.22 to produce the so-called 'Mohr envelope', which is curved. The figure shows that the Mohr failure criterion makes no provision for shear failure to be influenced by the intermediate principal stress.

It has become customary to combine the Mohr and the Coulomb failure criteria under the banner of the Mohr-Coulomb failure criterion by assuming a linear failure envelope as illustrated in Fig. 2.23. Equations 2.13, 2.14, 2.15, and 2.16 are some of the useful relationships that can be derived mathematically from this criterion, with a more extensive range to be found in Jaeger and Cook (1979) and Brady and Brown (2006).

$$\sigma_1 = \sigma_c + K\sigma_3 \tag{2.13}$$

where

$$K = \frac{1 + \sin\phi}{1 - \sin\phi} \tag{2.14}$$

$$\sigma_c = \frac{2c\cos\phi}{1 - \sin\phi} \tag{2.15}$$

$$= 2c\left\{\left[\sqrt{(Tan\,\phi)^2 + 1}\right] + Tan\,\phi\right\} \tag{2.16}$$

$$\sigma_T = \frac{2c\cos\phi}{1 + \sin\phi} \tag{2.17}$$

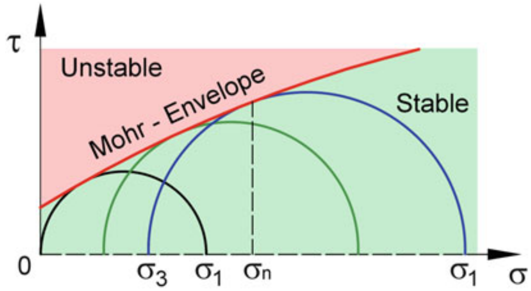


**Fig. 2.22** Mohr failure criterion

where

$\sigma_T = tensile\ strength$

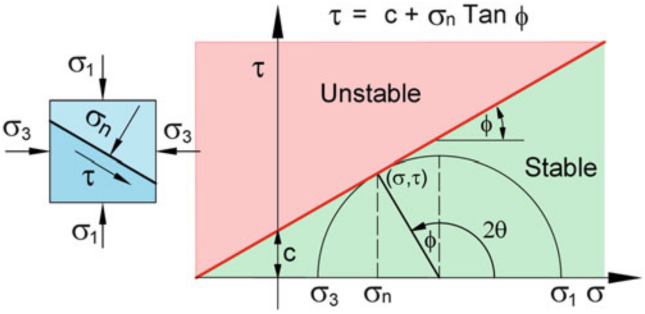
The angle of internal friction, ϕ, for coal typically ranges from 25° to 50° (Abel 1988; Salamon 1992a), giving a range for K of 2.5–5.5. This means that the strength of coal under triaxial compression will increase by 2.5–5.5 MPa for each 1 MPa increase in confining pressure.

The Mohr, Coulomb and Mohr-Coulomb failure criteria are simple and quick to apply and are utilised extensively in ground engineering. Nevertheless, a range of limitations and inaccuracies are associated with them. For example, the Mohr failure criterion is nothing more than a convenient way of expressing a critical stress condition. There is no proven evidence that at the point of failure, shear stresses exceed a critical value in a certain plane within the rock mass. Often failure does not coincide with the plane of failure derived from the strict application of Coulomb's or Mohr's failure criterion, and neither produces a realistic value for tensile strength.

An important point to appreciate in ground engineering is that shear displacement may have previously occurred on existing fracture planes. In these cases, so-called 'residual' values have to be substituted for cohesion and friction angle in failure criteria.

The Hoek-Brown strength criterion, developed in 1980, reduces the intact properties of rock to values that represent the in situ properties of the rock mass and permits the computation of both tensile and compressive strength (Hoek and Brown 1980). It describes a non-linear

**Fig. 2.23** Mohr-Coulomb failure criterion



relationship between confinement and stress, with the generalised version of the criterion, as at 2002, being given by Eq. 2.18 and described in detail by Hoek et al. (1995) and Hoek and Brown (1997).

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a \tag{2.18}$$

where

$\sigma_1'$, $\sigma_3'$ =*maximum and minimum effective principal stresses at failure*
$\sigma_{ci}$ =*intact compressive strength*
*GSI* =*Geological Strength Index*
$m_b$ =*a rock property, modified by the GSI and degree of disturbance of the rock mass*
$s$ =*a constant determined by the GSI and degree of disturbance of the rock mass*
$a$ = *a constant determined by the GSI*

The Hoek-Brown criterion is empirical and is neither new nor unique, with an identical equation used to describe the failure of concrete as early as 1936 (Hoek and Marinos 2007). According to Hoek et al. (1995), Hoek and Brown experimented with a number of distorted parabolic curves to find one that gave good coincidence with the original Griffith failure theory. The process used in deriving the criterion was one of pure trial and error. Apart from the conceptual starting point, there is no fundamental relationship between the empirical constants included in the criterion and any physical characteristics of the rock. The justification for choosing this particular criterion over the numerous alternatives lies in the adequacy of its predictions of observed rock fracture behaviour, and the convenience of its application to a range of typical engineering problems. As with the Mohr-Coulomb criterion, one of the acknowledged deficiencies of the Hoek-Brown strength criterion is that it does not allow for the influence of the intermediate principal stress on peak strength (Brown 2012a).

The significant contribution that Hoek and Brown made was to link the failure equation to geological observations. Bieniawski's Rock Mass Rating (RMR) was utilised for this purpose initially, before being superseded by the Geological Strength Index (GSI), both of which are discussed in more detail in Sect. 2.6.7. The Hoek-Brown failure criterion is based on field observation and testing and requires that the rock mass behaves isotropically and that failure does not follow a preferential direction imposed by the orientation of a specific discontinuity or a combination of two or three discontinuities (Marinos, Marinos, and Hoek 2005). Hence, it is only applicable to intact rock or heavily jointed rock masses which can be considered homogenous and isotropic. If two joint sets occur in a rock mass, the criterion can be used with extreme care, provided that neither of the joint sets has a dominant influence on the behaviour of the rock mass.

In 1997, Hoek and Brown reported that their failure criterion had been modified by others for application to a variety of rock masses, including very poor quality rocks, and adapted to meet the demands of software written in terms of the Mohr-Coulomb failure criterion. Results had been mixed and somewhat confusing, prompting them to set the record straight and to present an interpretation of the criterion which covers the complete range of rock mass types (Hoek and Brown 1997), Hoek (1998) and Brown (2008) provided further discussion on the reliability of Hoek-Brown estimates of rock mass properties and their impact on design.

Although in theoretical rock mechanics, the strength of materials is predominantly discussed in terms of stress, in practice most operating decisions as to the state of stability are based on displacements or associated strains. Displacement is a rock mass response that can be readily observed and interpreted in the field and accurately, easily and directly measured at multiple locations and converted into strain. Stress, on the other hand, is a derived quantity that cannot be quantified by observation in the field, is difficult, expensive and time consuming to measure, and prone to instrumentation inaccuracy and failure. Nevertheless, despite the emphasis and reliance placed on strain observations and measurements in operational situations, especially underground coal mining, strain based failure criteria have received only limited theoretical consideration.

An example of a failure criterion based on strain is that which can be derived from Eqs. 2.2 and 2.8, namely:

$$\varepsilon_l \geq \varepsilon_f \tag{2.19}$$

where

$$\varepsilon_f = critical\, failure\ strain,\ and$$
$$\varepsilon_l = -\nu\varepsilon_a = -\nu\frac{\sigma}{E}$$

Laboratory and field measurements indicate that many rocks will fail when lateral tensile strain approaches $1.5 \times 10^{-3}$, that is, 1.5 mm/m. This indirect tensile strain criterion is consistent with rock strength being higher for rocks that have a high elastic modulus and a low Poisson's ratio.

The stress and strain failure criteria discussed so far make no attempt to account for the cause of failure on an internal or microscopic scale. Griffith (1921) studied the crystalline structure of materials and hypothesised that the tensile stress distributions that develop at the ends of cracks cause them to propagate and to contribute to ultimate failure on a macroscopic scale. Griffith's failure criterion was developed on the basis of the conservation of energy and has been advanced over the years, notably by McClintock and Walsh (1962) and Murrell (1963).

In general, most failure criteria are simply hypothesis, rules of thumb or empirical constructions and tend to be popular because of their simplicity and versatility. The development of reliable failure criteria for rock structures is challenging because of the many factors that can influence rock strength and the difficulties in quantifying these factors. For example, none of the preceding failure criteria take account of anisotropy or the influence of the intermediate principal stress on the strength of jointed rock masses.

Because of size effect, laboratory based rock mass strength determinations are just estimates. Only the Hoek-Brown GSI rock mass strength model makes some quantitative strength assessment possible. The choice of scale is critical in assessing the rock fabric and should be related to excavation size. Typically, the volume of rock to be considered should be about three times the critical excavation dimension.

Hudson and Harrison (1997) list some of the many failure criteria that have been developed for rock. Judicious care is required in selecting constitutive laws and failure criteria best suited to a given situation and in interpreting and accepting the outcomes of analysis based on these laws and criteria. Some of these limitations have attempted to be addressed by relying on case studies and the concept of back-analysis to define stability limits. The reader is referred to Jing (2003) for a more comprehensive review of rock fracture models that find application in numerical modelling.

### 2.6.5 Effective Stress

In practice, the determination of shear strength in a mining environment is often far more complex than suggested by Eq. 2.11 because a range of natural and mining introduced factors can affect normal stress, cohesion and friction. When a normal stress, $\sigma_n$, is applied to a porous material or across a discontinuity, it will pressurise any fluid occupying the pores or the discontinuity unless the fluid can immediately dissipate. The fluid pressure is referred to as **pore pressure**, u, which, because it is hydrostatic, acts in all directions including opposite to that of the applied stress. Hence, fluid pressure negates a portion of the normal stress acting across contact points to clamp the discontinuity and prevent displacement. The reduced normal stress is referred to as **effective normal stress**, $\sigma_n'$. Under steady state conditions, where there is sufficient time for the water pressures in the rock mass to reach equilibrium, the effective normal stress is defined by Eq. 2.20.

$$\sigma_n' = \sigma_n \ \alpha_\mu u \tag{2.20}$$

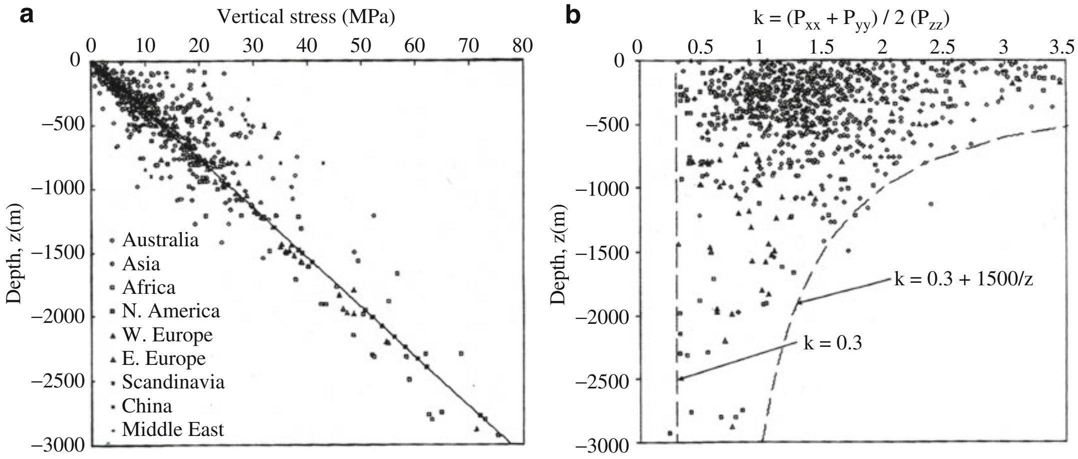


**Fig. 2.24** Variation of ratio of average horizontal stress to vertical stress with depth below surface (After Brady and Brown 2006 – data compiled by Windsor after Aydan and Kawamoto 1997)

where

$u\ =\ pore\ pressure$
$\sigma_n^{\prime}\ =\ effective\ normal\ stress$
$\alpha_\mu\ is\ usually\ 1,\ \ but\ may\ be\ <1$

In a highly permeable environment, pore pressure may be low and dissipate rapidly. However, it can be very high in moist materials of low permeability, such as clays, and in materials that are subjected to high confining stress. In these latter cases, the applied stress required to induce failure may be reduced significantly.

### 2.6.6 Primitive, Induced, Resultant and Field Stress

Prior to mining, all points in the Earth's crust are loaded by the mass of the overlying rock. The so-called **primitive vertical stress** (or **virgin vertical stress**), $\sigma_{vp}$, due to this overburden load is given by Eq. 2.21. Underground measurements and observations have revealed that the vertical direction does not always correspond to a principal stress direction when strata is dipping.

$$\sigma_{vp}\ =\ \rho g H \tag{2.21}$$

where

$\rho = overall\ density\ of\ overburden$
$g = gravitational\ acceleration$
$H = depth\ below\ surface$

Due to the Poisson's effect, the rock mass tries to expand laterally when subjected to primitive vertical stress. However, this is prevented by the confinement provided by the surrounding rock mass, resulting in the generation of a **primitive horizontal stress**, $\sigma_{hp}$, in the rock mass. Theoretically, the primitive horizontal stress should be a constant proportion, k, of the primitive vertical stress where $k = \nu/(1 - \nu)$. That is:

$$\sigma_{hp} = k\sigma_{vp} = \ k\rho g H = \left(\frac{\nu}{1-\nu}\right)\rho g H \tag{2.22}$$

A value for $\nu$ of the order of 0.25 is common for sedimentary rocks, in which case k should be 0.33; that is, theoretically, the horizontal primitive stress, $\sigma_{hp}$, should be about 1/3 of the vertical primitive stress and equal in all horizontal directions. However, at depths less than about 3,000 m, the horizontal primitive stress is rarely equal to about 0.3 times the vertical primitive stress nor equal in all horizontal directions, as evident in the data plotted in Fig. 2.24. Rather, the ratio of horizontal primitive stress to vertical primitive stress, k, can range up to 3.5 in one direction and down to about 0.6 in the orthogonal direction. The larger stress is referred to as the **major horizontal stress** and the smaller stress as the **minor horizontal stress**.

The effect of mining is to remove a portion of the rock mass through which the horizontal and vertical primitive stresses used to act. These stresses now have to deviate around the excavation, resulting in a change in the state of stress in the rock mass surrounding the excavation. The new state of stress is referred to as **resultant stress**. The difference between the primitive (original) stress and resultant (final) stress equates to the **mining-induced stress**. Other terms which find widespread use are **pre-mining stress** and **field stress**. These describe the state of stress already existing in rock that is going to be excavated. Both may or may not be equal to primitive stress, depending on whether the excavation site falls within the zone of influence of existing excavations.

The state of stress in rock is primarily comprised of two components, namely **lithostatic stress** generated by the weight of the overburden, and **tectonic stress** generated by movement of the continental plates. In times past, it was postulated that elevated horizontal stresses were lithostatic in nature, being due to erosion reducing the primitive vertical stress while locking in the primitive horizontal stress (see, for example, Voight 1966). However, subsequent analysis by Voight and St. Pierre (1974) and Haxby and Turcotte (1976) concluded that erosion can actually result in a reduction in horizontal stress because of the effects of uplift and contraction due to decrease in temperature. It is now accepted that tectonic forces are the primary contributing factor to elevated horizontal stresses, complemented by local features such as rapid changes in topography.

The primitive horizontal stress is usually expressed either as a proportion of the total vertical stress or as the sum of the lithostatic stress component (generated by the Poisson's effect) and the tectonic stress component. In the later case, the horizontal stress induced in a stratum by tectonic strain is often expressed as a function of the elastic modulus of the stratum, as shown in Eq. 2.23.

$$\sigma_{hp} = \left(\frac{\nu}{1-\nu}\right)\sigma_{vp} + TSF^{*}E_i \qquad (2.23)$$

where

*TSF* = *Tectonic Stress Factor* = *Tectonic induced strain*
$E_i$ =*Elastic modulus of* $i^{th}$ *stratum*

Equation 2.23 is regarded by many as unrealistic. The first term is based on assumptions of zero lateral strain during deposition of the strata and homogenous, isotropic, linear elastic behaviour. While this equation may be considered to apply for the initial condition, uplift, erosion and further deposition are likely to cause it to no longer apply (Brady and Brown 2006). Questions also arise as to what value of Poisson's ratio should be used in the equation for highly anisotropic and not necessarily linear elastic rocks. In relation to the second term, tectonic strain does not necessarily equate to plane strain.

The quantification of the in situ state of stress is complicated further by other factors including that the horizontal stress field is generally not transversely isotropic; very localised but high stress concentrations are associated with some fault and fold systems; and changes in stress magnitude and direction can occur around geological and topographical features. Hence, the calculation of primitive horizontal stress does not lend itself to an analytical approach and a high reliance has to be placed on in situ stress measurements and underground observations to quantify stress magnitudes and directions. This is particularly important in situations where escarpments and steep-sided valleys result in rapid changes in depth of cover and disruptions to stress trajectories, thereby causing local changes in the magnitude and direction of the principal horizontal and vertical stresses at the mining horizon.

Despite the effort that has been expended over the last 50 years in developing methods of in situ stress measurement and in understanding the factors that influence the in situ state of stress (see, for example, Fairhurst 2003, and Brady and Brown 2006), the reliable estimation of those stresses in any given case remains fraught with difficulty. This is particularly so in fissured coal measure rocks.

### 2.6.7 Field Stress in Coal

A number of characteristics are associated with coal that impact on stress magnitude within a coal seam and make the accurate determination of its state of stress problematic. These characteristics include:

- The modulus of coal is at the lower end of sedimentary rocks and therefore, in accordance with Eq. 2.23, a coal seam is subjected to less tectonic stress than rock types such as siltstone, sandstone and conglomerate which tend to comprise the surrounding strata.
- Coal has a porous structure and, therefore, a coal seam can have a capacity to sustain pore pressure.
- Many coal seams are cleated and jointed, with these fracture surfaces often extending into surrounding strata and having the potential to sustain fluid pressure.
- Cleated and jointed coal seams can act as aquifers, providing a source of fluid for the development of pore pressure.
- During the coalification process, a coal seam can absorb large amounts of gas that is held within the fabric of the coal.
- The formation in a coal seam of an excavation (which may range in size from a borehole through to a roadway) permits de-watering of the coal in the immediate vicinity and, therefore, a loss in pore pressure.
- A decrease in pore pressure increases effective stress in the coal which, subject to the stiffness of the loading system, can result in compaction.
- A decrease in pore pressure also reduces confinement to absorbed gas, resulting in some of this gas desorbing into the atmosphere.
- Coal shrinks as a result of gas desorption.

Virtually all stress measurement techniques result in a change in the state of stress that is trying to be measured. This is because excavation activities associated with accessing a measurement site result in a redistribution of stress about the immediate site. The situation is more complex when attempting stress measurements in coal because:

- The cleated and jointed nature of coal can interfere with the operation of stress measurement devices.
- Stress can change prior to and during measurement due to compaction associated with a reduction in pore pressure and to shrinkage associated with gas desorption.

These factors give rise to numerous permutations regarding the state of stress in coal, with each situation needing to be assessed in its own right. However, it can generally be stated that:

- Coal seams are not subjected to the high levels of horizontal stress of other rock types that typically comprise the immediate roof and floor of mine workings. Exceptions can arise in the absence of stiff surrounding strata, such as when lithologies comprise multiple coal seams and other low modulus rock types.
- The formation of a void in a coal seam results in a reduction or loss of any pore water pressure and, thus, an increase in effective stress. The impact of this on in situ stress magnitude and distribution is a function of the extent of depressurisation and the stiffness of the loading system. The extent of depressurisation is a function of permeability and time (see Sect. 10.3) and both are a function of the extent of extraction (see Sect. 4.3). If the superincumbent strata is sufficiently soft that it behaves as a deadweight load then it will subside and restore vertical stress to its pre-mining level. Otherwise, there may be a drop in vertical stress at the site of depressurisation and a corresponding increase in vertical stress remote from this site. Horizontal stress will increase due to the Poisson's effect as vertical stress recovers back to it pre-mining value.
- Desorption of gas from a coal seam results in a volumetric reduction in the fabric of the coal. This leads to a reduction in horizontal stress. It

may also result in a localised reduction in vertical stress, depending on the lateral extent of desorption and the stiffness of the surrounding strata.

A sound understanding of these behaviours is still in a state of development. Gale (2007) reported that field research at an Australian colliery had shown that over time, shrinkage of coal due to gas desorption caused a reversal in roof and rib extensometer movement such that rib coal moved back into the pillar and the roof moved upward. Because gas content in coal seams generally increases with depth, the effect of gas desorption on shrinkage could be expected to be greater at depth. Gale (2007) noted that coal roof conditions in poorly gas drained areas at depth at another Australian colliery were worse than in well drained areas. Coal roof conditions in drained areas were reported to be better than expected from stress calculations based only on lithostatic and tectonic stress. Gale concluded on the basis of his case studies that each one cubic metre of gas desorbed from a coal seam resulted in a 0.28 MPa reduction in in situ horizontal stress, which translated to improved roadway drivage conditions in coal at depth.

Against this background, the accurate measurement of pre-mining stress in coal is very problematic. The true state of stress is likely to have been altered by the creation of the void required to access the stress measurement site and by the associated reduction in pore pressure and increase in gas desorption afforded by the presence of the void. Amongst other things, these can produce a measured vertical stress that is less than that corresponding to overburden load and a range of measured horizontal stress values that are a function of time. The reader is referred to the literature, including Enever et al. (2000), Shen et al. (2003), Aziz et al. (2005) and Gale (2007) for a fuller discussion on this subject.

### 2.6.8 Field Shear Strength

When intact rock is subjected to shear, shear stress increases steeply until the peak shear strength is reached and sliding is initiated on the fracture plane. As displacement continues, the surfaces of a fracture plane are 'polished', resulting in shear strength reducing to some residual value that is determined by residual cohesion, $c_r$ and residual friction angle, $\phi_r$. For most rock surfaces, the value of residual cohesion is small, being of the order of 0.1 MPa (Cook et al. 1974) and, for practical purposes, is often taken to be zero.

Residual shear strength is of great significance in ground engineering because, unlike in most other branches of engineering, structures have to be formed and remain functional and safe in materials that already contain failure surfaces and which are subjected to new failures as a result of mining-induced stresses. Pre and post-failure analysis is complicated by factors such as the presence of fluids, irregular surface profiles of natural discontinuities, infill on discontinuities, and the installation of reinforcement across fracture planes.

Undulations on natural discontinuities function as a form of mechanical interlock as illustrated in Fig. 2.25. For shear displacement to occur, the opposing surfaces either have to ride up over each other, causing the rock mass to dilate (increase in volume), or else break off. The former is equivalent to additional frictional resistance and the latter to an additional component of intact cohesion that is determined by the contact strength of the rock comprising the surface undulations. Based on laboratory experimentation, Patton (1966) proposed that Eq. 2.24 could be used to account for the frictional resistance of a "saw tooth" joint surface. Hoek et al. (1995) report that this equation is valid at low normal stresses, where shear displacement is due to sliding along the inclined surfaces. However, at higher normal stresses, the strength of the intact material will be exceeded and the teeth will tend to break off, resulting in shear strength behaviour which is more closely related to the intact material strength than to the frictional characteristics of the surface.

$$\tau_r = \sigma_n tan\,(\phi_b + i) \tag{2.24}$$

where

$\tau_r =$ *residual shear strength*
$\phi_b =$ *basic friction angle*
$i =$ *angle of saw tooth face*

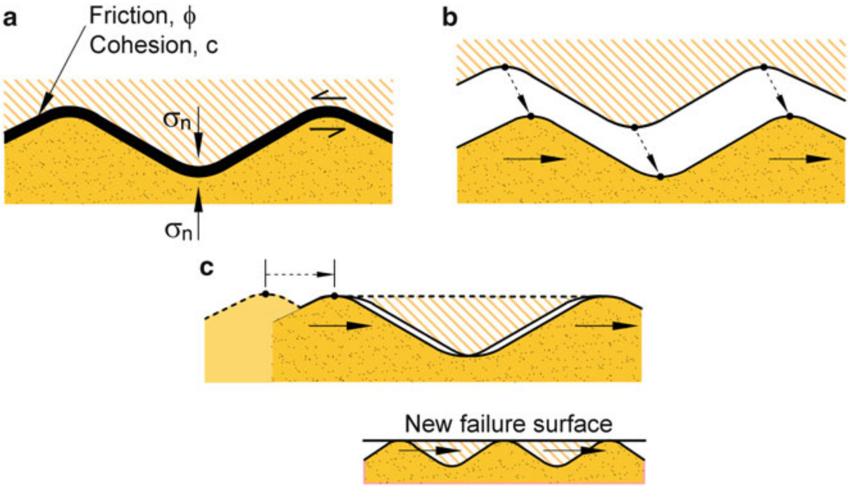


**Fig. 2.25** Mechanical interlock component of shear resistance. (**a**) No relative displacement between beds. Contact surfaces locked together by cohesion, friction and resistance of rock contact surfaces to failure under shear stress. (**b**) Relative displacement between beds due to beds separating and riding over rock contact surfaces. (**c**) Relative displacement between beds due to contact strength of the rock surfaces being exceeded (After Galvin et al. 1994)

Barton (1973, 1976) developed this concept by recompiling Eq. 2.24 in the form of a relationship based on joint roughness and joint wall compressive strength, Eq. 2.25. This has undergone further refinement by Barton and Choubey (1977) to account for the residual friction of weathered rock and now features in the Barton-Bandis criterion for rock joint strength and deformability (Barton and Bandis 1990). The reader is referred to Hoek et al. (1995) for further detail.

$$\tau_r = \sigma_n tan\left(\phi_b + JRC.log_{10}\left(\frac{JCS}{\sigma_n}\right)\right) \quad (2.25)$$

where

$JRC = joint\ roughness\ coefficient$
$JCS = joint\ wall\ compressive\ strength$

Shear strength can be reduced significantly when shear surfaces contain soft infill material or fluid. If the thickness of the infill exceeds the amplitude of surface undulations, the shear resistance of the discontinuity will be determined by the cohesive and frictional properties of the infill rather than the rock surfaces. In addition to reducing effective normal stress, in some cases the presence of fluid can lubricate the surfaces and reduce the frictional component of shear resistance.

### 2.6.9 Reduction in Confinement

Earlier in this chapter and permeated throughout this text, there is a focus on the beneficial effects of increasing confinement to improve rock mass strength and on the capacity of rock to sustain an increase in compressive loading stress. This is a reflection of the emphasis traditionally placed in underground mining on stress increase in the rock mass and the implications of this for mine stability. It is consistent with standard laboratory test procedures for determining the mechanical characteristics of a rock specimen, where the specimen is confined laterally and then subjected to increasing axial load.

However, the loading process in the underground mining environment is almost the direct reverse of these situations. The rock mass is already under considerable load prior to mining, with mining removing confinement to the pre-loaded rock mass around the perimeter of the excavations. At the same time, mining also results in an increase in load in the abutments of

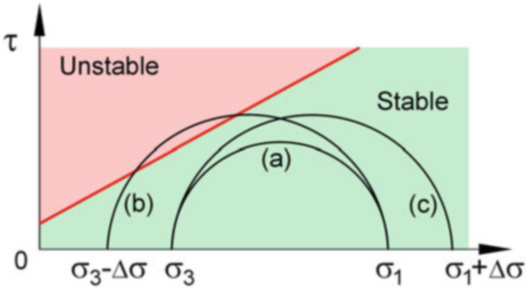


**Fig. 2.26** Mohr-Coulomb diagram illustrating the comparative effect on rock stability of reducing (lateral) confining stress by a given amount, Δσ, as compared to increasing (axial) stress by the same amount

the excavation. The magnitude of this load increase is a function of a range of factors discussed in later chapters. The important point to appreciate is that stress reductions associated with loss of confinement when excavations are formed in the rock mass may have a greater impact on the state of stability than corresponding increases in loading stress.

The comparative effect on rock stability of reducing confinement as opposed to increasing load can be illustrated with the aid of a Mohr-Coulomb diagram, as shown in Fig. 2.26. In this example, the rock mass is assumed to be coal, with $\sigma_3$ the confining stress and equal to $0.375\sigma_1$ ($k = 3/8$). Curve (a) is constructed on the basis of $\sigma_1$ and $\sigma_3$. Curve (b) is based on $\sigma_1$ and an incremental reduction in $\sigma_3$ of $\Delta\sigma$. Curve (c) is based on holding $\sigma_3$ constant and increasing $\sigma_1$ by $\Delta\sigma$. Hence, the deviator stress associated with curves (b) and (c) is the same. However, the impact on the state of stability is markedly different, with the incremental reduction in confinement moving the system closer to instability than the same incremental increase in loading stress.

### 2.6.10 Rock Mass Classification Systems

Conventional rock mass classification systems are scoring schemes which attempt to characterise the quality or competence of a rock mass by assigning a numerical rating to factors thought to affect the stability or behaviour of the rock mass, and summing these to produce a single numerical index. A number of secondary schemes have been developed which endeavour to correlate rock mass indices with field experience in areas such as excavation span, stand up time, ground support requirements, pillar safety factor, caveability, fragmentation, and slope stability. These types of approaches are not mechanistically based and have been superseded to some extent by the advent of powerful numerical analysis methods, prompting the development of modified classification systems which provide rock mass ratings that are more useful as input into these models.

A large number of rock mass classification systems have been developed since the mid 1960s. These have focused mainly on the civil engineering tunnelling sector, although some have been applied to mining operations, predominantly in the hard rock sector. One of the earliest and simplest is the Rock Quality Designation, or RQD, proposed by Deere (1964). The RQD is defined as the percentage of core recovered as intact pieces of 100 mm or more in length by diamond drilling.

The Geomechanics Classification System, also known as the Rock Mass Rating System (RMR), and the Tunnelling Quality Index (Q System) are the most widely utilised rock mass classification schemes. The RMR System was developed by Bieniawski (1974) for the tunnelling industry. The RMR Index ranges from 0 to 100 and is calculated by adding the individual ratings for six factors, (Table 2.6). Table 2.7 summarises the rock mass description and class associated with the resulting RMR Index.

A more detailed description of the system can be found in Bieniawski (1989). It has been adapted in a number of ways to the mining industry, two of the better-known examples being the Mining Rock Mass Rating System (MRMR) as described by Laubscher (1990) and the Coal Mine Roof Rating (CMRR) as described by Mark et al. (2002).

The Q System was developed by Barton et al. (1974) and is based on the formulation described by Eq. 2.26.

**Table 2.6** Factors and scoring ranges associated with calculation of the RMR Index

| Factor | Range |
|---|---|
| Uniaxial Compressive Strength | 0–15 |
| Rock Quality Designation (RQD) | 3–20 |
| Spacing of Discontinuities | 5–20 |
| Condition of Discontinuities | 0–30 |
| Orientation of Discontinuities | 0–15 |
| Groundwater | −12–0 |

**Table 2.7** Rock mass description and class associated with RMR Index

| RMR index | Description | Rock mass class |
|---|---|---|
| 81–100 | Very good rock | I |
| 61–80 | Good rock | II |
| 41–60 | Fair rock | III |
| 21–40 | Poor rock | IV |
| <20 | Very poor rock | V |

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \qquad (2.26)$$

where

$Q = Rock\ Tunnelling\ Quality\ Index$
$RQD = Rock\ Quality\ Designation$
$J_n = Joint\ Set\ Number$
$J_r = Joint\ Roughness\ Number$
$J_a = Joint\ Alteration\ Number$
$J_w = Joint\ Water\ Reduction\ Factor$
$SRF = Stress\ Reduction\ Factor$

This formulation produces an index in the range of 0.001–1000, which is ranked on a logarithmic scale to produce nine rock quality classes. Further details of the Q System, the RMR rock mass classification schemes, their derivatives, applicability and reliability are to be found in Hoek et al. (1995), Mikula and Lee (2003), MCA (2003) and Palmstron and Broch (2006).

The development of sophisticated and powerful numerical modelling methods created a need for more reliable information on the in situ strength and deformation characteristics of rock masses. The Hoek-Brown failure criterion has found widespread application in this regard. Initially, the criterion relied on the input of selective values determined using the RMR system. However, this was found wanting and in the early 1990s, Hoek and his co-workers set about developing the Geological Strength Index (GSI) in order to estimate the reduction in rock mass strength due to geological conditions (Hoek, Wood, and Shah 1992; Hoek 1994). This has been progressively refined to account for weaker, heterogeneous rock masses (Marinos and Hoek 2001).

The GSI classification system is based on the assumption that the rock mass contains a sufficient number of randomly oriented discontinuities to cause it to behave as an isotropic mass. It is essentially qualitative because it was considered that the numbers associated with the RMR and Q Systems were largely meaningless for weak and heterogeneous rock masses (Marinos et al. 2005). The geological characteristics of rock material and a visual assessment of the rock mass that this material forms are used to determine the value of the GSI (Fig. 2.27). This value is entered into a set of empirically developed equations to estimate the rock mass properties, including friction, cohesion, and modulus. The approach enables a rock mass to be considered as a mechanical continuum while still taking into account the influence that geology has on its mechanical properties.

It is important to appreciate that the GSI is based on the assumption that the rock mass contains a sufficient number of randomly orientated discontinuities for it to behave as an isotropic mass; that is, the behaviour of the rock mass is independent of the direction of the applied load. Furthermore, it is only concerned with the estimation of rock mass properties. It does not seek to provide a rock mass reinforcement or support design capability. The classification system finds application in a range of empirical procedures, such as that proposed by Hoek and Diederichs (2006b) for estimating rock mass modulus, as do the RMR and Q systems (see Bieniawski 2011).

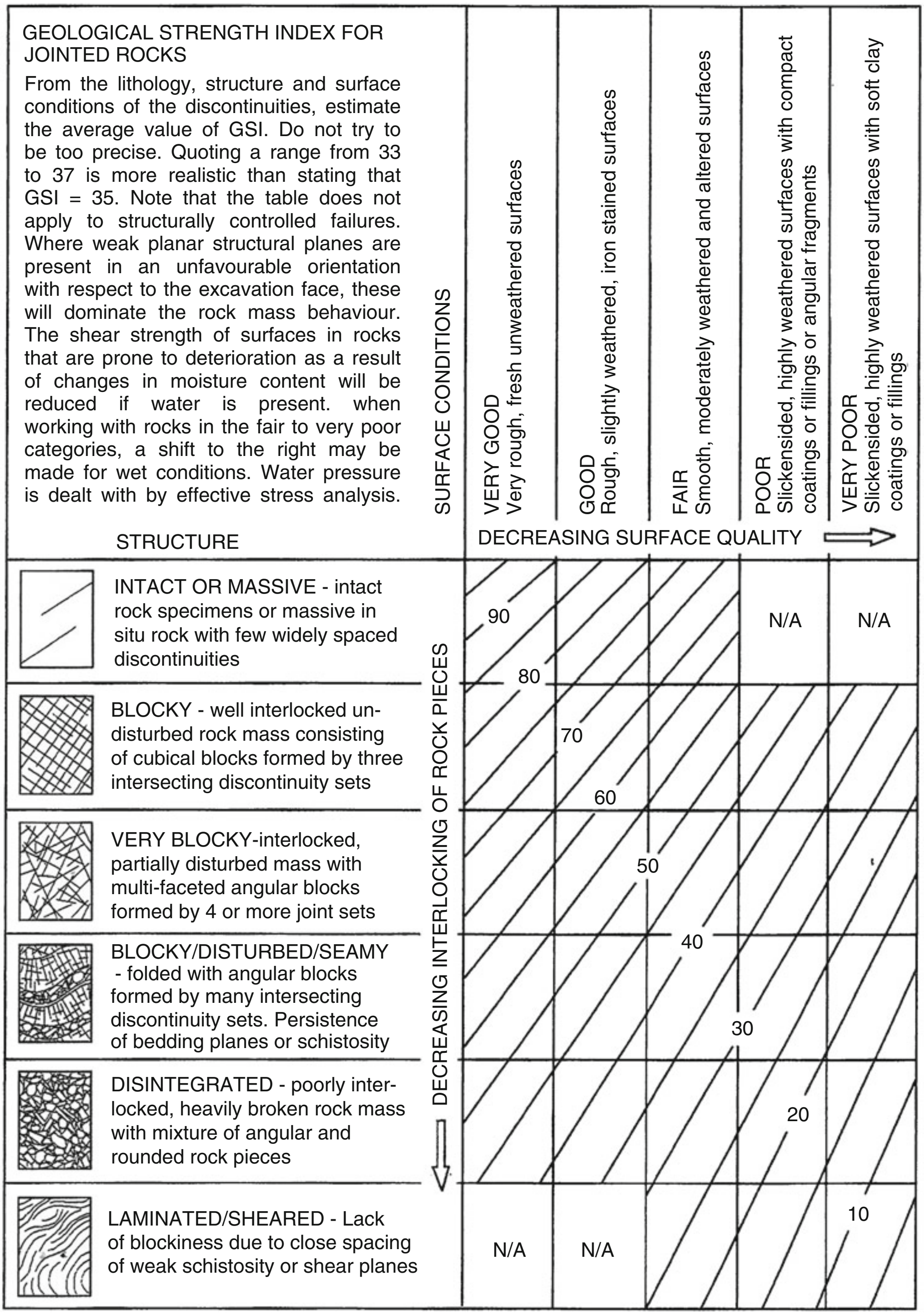

GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS
From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavourable orientation with respect to the excavation face, these will dominate the rock mass behaviour. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced if water is present. when working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis.
SURFACE CONDITIONS
VERY GOOD
Very rough, fresh unweathered surfaces
GOOD
Rough, slightly weathered, iron stained surfaces
FAIR
Smooth, moderately weathered and altered surfaces
POOR
Slickensided, highly weathered surfaces with compact coatings or fillings or angular fragments
VERY POOR
Slickensided, highly weathered surfaces with soft clay coatings or fillings
STRUCTURE
DECREASING SURFACE QUALITY
INTACT OR MASSIVE - intact rock specimens or massive in situ rock with few widely spaced discontinuities
BLOCKY - well interlocked un-disturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets
VERY BLOCKY-interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets
BLOCKY/DISTURBED/SEAMY - folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity
DISINTEGRATED - poorly inter-locked, heavily broken rock mass with mixture of angular and rounded rock pieces
LAMINATED/SHEARED - Lack of blockiness due to close spacing of weak schistosity or shear planes
DECREASING INTERLOCKING OF ROCK PIECES
90
80
70
60
50
40
30
20
10
N/A
N/A
N/A
N/A

**Fig. 2.27** Geological Strength Index (GSI) classification system for jointed rocks (Courtesy of Professor Evert Hoek)

The Coal Mine Roof Rating (CMRR) is intended to evaluate the roof discontinuities which most contribute to the weakness and failure of the roof mass (Molinda, Mark, and Debasis 2001). The rating system was designed to be equivalent to the RMR so that the CMRR/unsupported span/stand-up time relationship is nearly the same in both systems. The CMRR is based on data collected from USA mines and produces an index in the range of 0–100 derived from:

- fracture spacing and frequency, or RQD;
- uniaxial compressive strength derived from laboratory testing, axial point load testing, or indentation depth of a ball peen;
- tensile strength of bedding and discontinuities determined by diametric point load testing; and
- thickness weighted average of strata units.

This data is collected from underground exposures such as roof falls and overcast cut-outs and from drill core. The CMRR has been incorporated into aspects of coal mine planning, including gateroad pillar design, determination of roof bolt length and support density utilising logistic regression (discussed in Sect. 2.7.5). Limitations are associated with these applications since, as with all rock mass classification systems, the CMRR does not take account of behaviour mechanisms.

Two of the most basic rock mass classification schemes are the Roof Strength Index (RSI) and the Stress Strength Ratio (SSR). The RSI is the ratio of uniaxial compressive strength (UCS) to depth and the SSR is the ratio of depth to CMRR. Both equate to a primitive form of safety factor pertaining to the immediate roof strata, with depth corresponding only to virgin stress, not total stress, and UCS and CMRR having some relationship to the rock mass strength of the immediate roof. RSI is reported to have worked well as a predictor of poor roof conditions at Kestrel Mine in Australia where it was developed, and at the neighbouring Crinum Mine (Gordon and Tembo 2005; Payne 2008; and Gordon 2009). This may be because these mines have consistently weak roof and floor, relatively benign mining conditions, and repetitive mine layouts, thus resulting in uniform and uncomplicated conditions.

The Geophysical Strata Rating (GSR), developed by Hatherly et al. (2008), estimates the quality of rock masses based on the analysis of geophysical logging data. Its main advantages over alternative rock mass classification systems are claimed to be the objectivity of using geophysical logs, the ability to run GSR over the entire length of a borehole, and the potential to conduct studies based on historical borehole log data. The developers report that in rocks of reasonable quality, GSR values are similar to those obtained with the CMRR. In poorer quality rocks, particularly those with high clay content, relatively high porosities and low sonic velocities, GSR values are lower than CMMR values. The method is claimed to provide for the discrimination of very weak, clay rich units which do not contain obvious defects and which are difficult to characterise by the CMRR alone.

Rock mass classification schemes which endeavour to encapsulate the complexity and diversity of a natural rock mass in a single numerical index are attractive and offer advantages because of their simplicity; because they cause rock mass properties to be evaluated in a systematic and continuous manner; and because they can be calibrated to previous experience. However, they have shortcomings and must be used with care. In general:

- Most rock mass classification systems give little or no consideration to:
  - the characteristics of the surrounding rock mass;
  - impacts which might arise from deformation and mobilisation of the surrounding strata during mining;
  - single geological features, such as an unfavourably orientated plane of weakness or a thin stratum with poor mechanical properties, the behaviour of which is the dominant factor causing structural failure;
  - stress anisotropy;
  - the influence of mining direction.
- Not all of the critical factors which control ground response in a mining environment may be incorporated into the rating system; for example, the number of joint sets or the dip of joint sets.

- Adjustments which are made to account for the influence of mining are often of a subjective nature.
- The systems are suited primarily to situations where failure is controlled by sliding and rotation of intact pieces of rock at low to moderate stress levels. They do not cater for situations where failure is associated with squeezing, swelling, spalling or pressure bursts, or where failure develops progressively.
- The numerical value of the resulting rock mass index can be highly dependent on the local knowledge and experience of the person assessing the rock mass.

Hence, rock mass behaviour mechanisms and failure modes are largely ignored in rock mass classification systems and all important controlling aspects may not be fully evaluated. Two quite different rock mass structural settings or rock mass behaviour mechanisms, for example, can have the same rock mass classification index. Similarly, a change in a critical factor will not be reflected in a classification index unless this factor is explicitly included in the classification rating scheme. Caution is required, particularly with design procedures that rely on direct correlations to rock mass ratings.

Hoek and Brown (1980) pointed out the dangers involved in blindly adopting the provisions of the Q system. Hoek et al. (1995) emphasised the importance of understanding that the use of a rock mass classification scheme does not (and cannot) replace some of the more elaborate design procedures. This advice was reinforced by Hartman and Handley (2002), stating that it must be understood that a classification system can give the guidelines but the geologist or engineer must interpret the finer details. Brady and Brown (2006) noted that while a rock mass classification approach is superficially attractive, it has a number of serious shortcomings and must be used with extreme care. It does not always fully evaluate the important aspects of a problem and, if applied blindly without supporting analysis of the mechanics of the problem, can lead to disastrous results.

Subsequently, Pells (2008) expressed concern at the inappropriate and sometimes dangerous manner in which rock mass classification systems are used to quantify behaviour. Pells was particularly critical of the design of tendon support systems on the basis of rock mass classification systems, noting that they provide little or no idea of the loads that the reinforcement is supposed to carry or the shear and tensile displacements the bolts are expected to encounter. Bieniawski (2011) reported that he has always advocated that rock mass classification systems should always be used in conjunction with computer modelling and field monitoring of performance but, by the same argument, they should not be dismissed from the process of design as they play a critical role in rock mass characterisation, bridging qualitative geological description in quantitative engineering data. Suorineni (2014) issued a number of cautions in regard to the databases that underpin some classification systems and their scope of application.

Some rock mass classification systems are no longer just being used as a point of reference to past outcomes but also as a primary determinant of mechanism of behaviour. In underground coal mining, examples are to be found in ground support design and pillar system design. By way of illustration, the determination of roof support patterns is critically dependent on the orientation of joint systems, the orientation and magnitude of horizontal stress, the direction of drivage, and the presence of very weak individual units, none of which feature in the derivation of rock mass ratings on which some support designs are based.

The issue is laboured because of the risks that can be associated with the inappropriate use of rock mass classification systems in underground mining, especially when used as a basis for mine design and operating procedures. These systems are not mechanistically based or rigorous and, accordingly, it is advisable that they are not correlated directly to mechanisms of behaviour or used in isolation.

### 2.6.11 Failure Mode

Rock can be in a state of equilibrium under one of two forms, namely **stable equilibrium** and **unstable equilibrium**. These can be visualised as shown in Fig. 2.28. When a system is in a state of stable equilibrium, energy has to be put into the system to change this state. On the other hand, the slightest disturbance to a system that is in a state of unstable equilibrium results in the sudden release of energy from the system.

In order for rock to begin to deform, external energy has to be put into the system. Up to the point of maximum resistance to deformation, part of this energy is used to create fractures while most of the remainder is stored in the system in the form of strain energy. Once the point of peak stress (that is, strength) is exceeded, resistance to deformation decreases and the stored strain energy is available to drive further deformation of the rock structure and to create additional fractures. If this energy is insufficient to cause further deformation and fracturing, then the system will stabilise. Otherwise, the deformation process will become unstable and the rock structure will rupture violently.

In order to assess the mode of structural failure of a rock structure, both the energy required for rock deformation and rock fracturing and the energy stored in the system need to be known. The energy stored in the system during loading to the point of maximum resistance to deformation depends on the stiffness of the system. The softer the system, the greater the amount of energy stored within it (under a given load) and, consequently, the greater the amount of energy available to drive post-peak strength deformation of the rock structure.

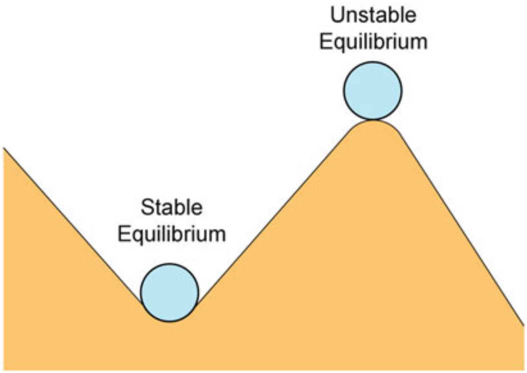


**Fig. 2.28** Visualisation of stable and unstable states of equilibrium

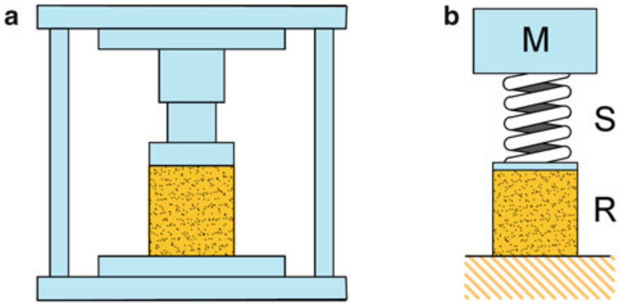
The principles involved can be explained by considering a rock specimen that is loaded in a compression testing machine, illustrated in Fig. 2.29a. The specimen behaves as a spring and exerts an equal and opposite force on the testing platens to that which the platens exert on it. The testing machine can be visualised as a mass, M, acting in series with a spring, S, that represents the stiffness of the testing machine (Fig. 2.29b). The stiffer the testing machine spring, the less it will compress under the reaction load of the specimen and therefore, in accordance with Eq. 2.6, the less potential energy that will be stored in it.

The stiffness, or deformation characteristics, of the testing machine spring can be determined by replacing the specimen with a hydraulic jack, lowering the mass M by a given distance, $d_1$, and then plotting the pressure in the jack against the position of the mass as the mass is jacked back to its original position. This produces the line labelled $l_1$ in Fig. 2.30. If this process is repeated a number of times, with the mass being lowered a greater initial distance each time, a series of parallel load-displacement lines, known as loading lines, will be produced. The stiffer the loading system, the steeper the loading lines.

If the rock specimen is now placed in the testing machine instead of the hydraulic jack and loaded, one of two conditions can arise. Firstly, the loading lines of the machine can intersect the load-displacement curve of the specimen over its full loading cycle as depicted in Fig. 2.31a. In this case, the loading machine can be stopped at any time and the system will remain stable, irrespective of the state of the specimen. At all times, the specimen retains sufficient stiffness to prevent the release of the potential energy stored in the spring of the testing machine.

Alternatively, the condition can arise where, as a result of progressive failure, the slope of the load-displacement curve for the specimen

**Fig. 2.29** Model for simulating the loading of a rock specimen (After Salamon and Oravecz 1976)



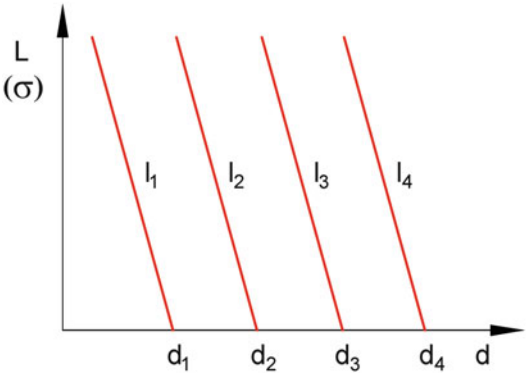


**Fig. 2.30** Loading lines for a rock testing machine (Adapted from Salamon and Oravecz 1976)

becomes steeper than that of the loading lines of the compression machine. This occurs at point F in Fig. 2.31b. The instant that this situation is reached, the stiffness of the specimen is no longer adequate to resist the release of the energy stored in the loading system. Consequently, the potential energy in the loading machine spring will be converted to kinetic energy, causing the mass M to accelerate suddenly and impart this energy to the specimen to result in explosive failure. This process may be assisted by strain energy stored in the specimen.

The significance of rock failure occurring in a controlled manner is that rock still maintains a substantial load carrying capacity after its maximum resistance to deformation has been exceeded. This has major implications for mine design and safety. Note that stress-strain plots can be substituted for load-displacement plots in Fig. 2.31a, b because they mirror the same response.

### 2.6.12 Ground Response Curve

A so-called ‘**ground response curve**’, such as that shown in Fig. 2.32, provides a convenient means for conceptualising the interaction between the rock mass and ground support systems when an excavation is formed in the rock mass. Prior to creating the excavation, the rock mass within the outline of the excavation provides the support resistance and so there is no convergence (point A). As the opening is created, there is a decrease in support resistance and convergence occurs, representing the elastic response (elastic rebound) of the rock mass. It is impractical for reasons of timing and support capacity to install support to prevent this convergence.

With further increase in excavation span, a point is reached where the ground response curve becomes non-linear and starts to flatten (point B) due to a reduction in the self-supporting capacity of the surrounding rock mass as it fractures and yields. At some stage, it becomes practical to install support (point S), which is then loaded by the ongoing convergence. The load capacity and yielding characteristics of this support determine if and when equilibrium is reached between the support and the rock mass (point E). In the case of the roof strata, failure to control convergence can ultimately lead to an increase in the required support resistance due

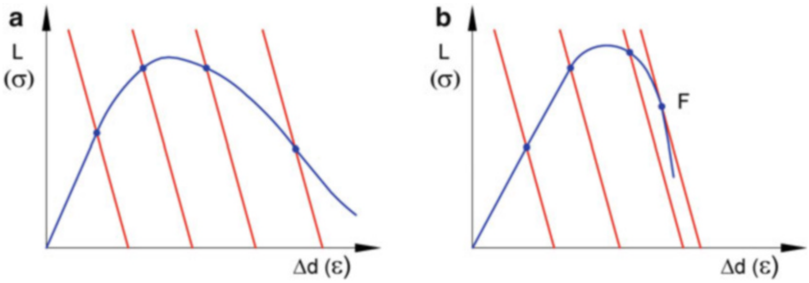


**Fig. 2.31** Loading lines and stress-strain curves associated with controlled and uncontrolled rock failure (**a**) Controlled failure (**b**) Uncontrolled failure (Adapted from Salamon and Oravecz 1976)

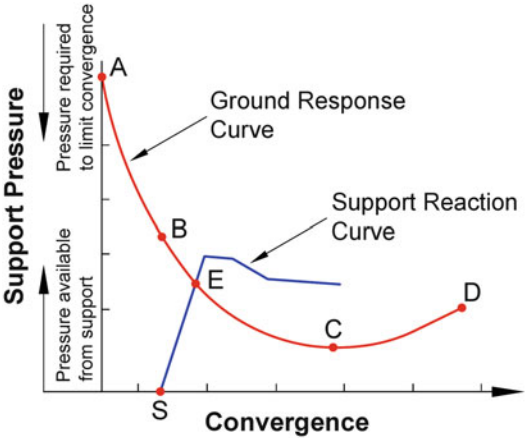


**Fig. 2.32** Ground response and support reaction curves (Adapted from Daemen 1977)

to the increased height of loosening of fractured strata and the transfer of its deadweight to the support system (section C to D).

## 2.7 Analysis Techniques

The fundamental principles presented so far in this text should already make it apparent that the analysis of rock mass response to underground excavations can be complex, invoking a number of core disciplines including geology, engineering geology, fluid mechanics, soil mechanics, and rock mechanics in circumstances where:

- the rock mass may consist of many different rock types of widely different material properties;
- the material comprising each rock type is often of variable composition and anisotropic;
- laboratory determined material properties do not directly reflect the in situ rock mass properties;
- rock mass strength is influenced by fluid characteristics and pressures;
- local and regional structural properties and in situ stress fields affect rock mass response;
- economic considerations limit the amount of detailed in situ sampling and measurement that can be undertaken to characterise the rock mass prior to mining, with the budget for mine site investigations typically being several orders of magnitude less than in civil engineering circumstances;
- rock behaviour is governed by complex and variable constitutive laws. For example, no rocks appear to exist that obey Hooke's Law or even some generalised linear hereditary law under all conditions. However, virtually all rocks comply with linear constitutive laws under some conditions (Salamon 1988).

Hence, rock mechanics problems are data limited problems that cannot be modelled unambiguously (Starfield and Cundall 1988), necessitating a range of assumptions regarding rock mass properties and system behaviour. Irrespective of the analysis technique, a degree of uncertainty is associated with model selection, input parameter values and human error.

It is convenient and common in all types of analysis techniques to express some parameters in the form of a dimensionless ratio, in a process referred to as '**normalising**'. The ratio of

**excavation width**, W, to **mining depth**, H, is an example of such a ratio. Care is required when utilising these types of expressions as the sensible bounds of the relationship may not be readily apparent. For example, sub-surface and surface subsidence behaviour above a panel with a W/H ratio of 1 at a depth of 50 m is likely to be very different to that over a panel with the same W/H ratio at a depth of 350 m.

Given the uncertainty associated with all geotechnical analysis and design, reliance has to be placed on judgements. These should be premised on knowledge and experience. Risk assessment and independent peer review are important controls for managing this uncertainty. This section provides a summary review of the major categories of analysis techniques that support design, judgments and risk assessment.

### 2.7.1 Empirical Methods

Empirical analysis is concerned with evaluating and predicting behaviour on the basis of experiment, field data and observation. A pure empirical approach to analysis is one based on a series of controlled experiments in which the influence of each variable is examined in turn (Salamon 1974). However, in most instances in ground engineering in underground mining, it is not possible or practical to perform a sufficient number of experiments or to analyse a real engineering problem exhaustively in terms of all possible variables in order to obtain quantitative general solutions. This is addressed by adopting a scientific approach to empirical research that is focussed on only investigating the effects of the most important or primary variables. Success is dependent on identifying all of these variables and having a database which contains sufficient relevant information to evaluate the influence of them (Salamon 1992b, 1993).

This empirical research based approach contrasts with the other extreme of developing relationships between parameters on the basis of trial and error and curve fitting that has little regard to whether the relationships so developed are mechanistically valid. That approach is fraught with risk and does not constitute sound empirical analysis. Unfortunately, the advent of spreadsheet plotting software supported with inbuilt statistical routines for undertaking regression analysis and for assigning mathematical equations to relationships has seen a proliferation of the approach. It would appear that some practitioners are oblivious that the plotted parameters often have no physical relationship in reality or that the statistical confidence levels quoted for these contrived relationships sometimes indicate that the relationships are, in fact, unsound.

Some advocates of empirical analysis based on curve fitting to data place weight on the words of Salamon (1989) that the main advantage of empirical analysis *“is its firm links to actual experience. Thus, if it is judiciously applied, it can hardly result in a totally wrong answer. Also, in our legalistic world, it has the added advantage of defensibility in a court of law. After all, it is based on actual happenings and is not just a figment of imagination.”* These words need to be understood in context, noting that they were qualified with *‘if judiciously applied’*. In the same publication Salamon stressed that effective back-calculation requires a reasonably clear understanding of the physical phenomenon in question and that this is a feature that distinguishes it from ordinary linear or non-linear regression used in statistics.

Extrapolation of empirical relationships is always fraught with risk. Considerable care has to be exercised if applying these types of relationships outside the range of the data used in their derivation or to ‘greenfield’ sites. Advances in numerical analysis are providing more reliable insight into the mechanistic relationships between parameters and their relative influence on ground behaviour, hence resulting in some applications of empirical analysis becoming obsolete. In an attempt to keep up with evolving knowledge and to produce reliable outcomes, some empirically based techniques have undergone refinements by way of introducing additional assumptions and calibration factors, with many of these weakening an already tenuous link to the mechanics of the real behaviour.

Empirical approaches which disregard the mechanics of behaviour and instead, rely on subjecting databases to simple statistical correlations such as linear regression are not scientific, regardless of the effort and care that has gone into collecting and plotting the data. To properly use empirical methods, one must understand the underlying assumptions and the databases used for their development (Suorineni 2014). Given this and a reasonably clear understanding of the underlying physical phenomenon, they can form the bases of valuable design tools.

### 2.7.2 Analytical Methods

Analytical methods utilise mathematical solutions derived from first principles of physics and mechanics to determine stress and displacement responses. The methods rely on selecting the correct mode of behaviour and incorporating the key variables that control this behaviour. Generally, these types of solutions are restricted to the analysis of pre-failure behaviour in two-dimensional situations, such as displacement distributions within a beam or plate or stress distributions around a long excavation of a specific cross-sectional shape, such as depicted in Fig. 2.13. The solutions find particular application in mining geomechanics when a structure is long and continuous in the third dimension, such as in the case of a tunnel, because the analysis can then be based on a two-dimensional cross-section.

Analytical techniques tend to be quick and low cost and can produce quite accurate outcomes, especially in circumstances where the surrounding rock mass conditions are reasonably homogenous and have been accurately characterised. They give insight into fundamental physical principles and are useful for undertaking parametric and sensitivity analysis to identify critical behaviour modes that require more in-depth analysis.

Care is required with the methodology when failure may involve one or more modes that need to be analysed incrementally and simultaneously to determine stress paths. This is a complex process that is constrained further by imprecise input parameters and computational capacity. A technique often employed to simplify such solutions is to equate the surrounding strata to a single stratum of ‘equivalent’ material properties. Appendix 2 provides an example of this approach as applied to elastic modulus. The advent of the computer has enhanced the scope to apply analytical solutions to general mining situations and many solutions now constitute the basis of so-called **numerical analysis**, **computer modelling**, and **mathematical modelling**.

### 2.7.3 Numerical Methods

Numerical methods are concerned with simulating rock mass behaviour using mathematical equations founded on analytically derived formulae, or algorithms. Most numerical analysis in relation to geotechnical engineering involves utilising computers to solve vast arrays of simultaneous equations using iterative and approximate techniques. Hence, numerical analysis is also referred to as mathematical modelling and is effectively a powerful form of analytical analysis.

Salamon (1989) reported that he came to the conclusion in the late 1950s that numerical modelling is essential in strata control because the number of variables is so great that it is entirely impractical to explore experimentally their full range of influences. At the same time, no mathematical model is sufficiently general or complete to incorporate all physical aspects of the rock mass, its behaviour and the geometry, support etc of the mine. Thus, field experiments are vital in the evaluation of the efficacy of the models. Salamon’s PhD thesis, submitted in 1962, appears to have contained the first proposal for numerical analysis on the basis of mathematical models, with Salamon lamenting in 1989 that it was frustrating to watch the reluctance on the part of operators and even specialists to accept and pursue modelling opportunities (Salamon 1989). Unfortunately, this reluctance persists in some quarters.

A range of computer codes, or numerical models, have been developed to simulate rock behaviour, with these falling into two main classes:

- Boundary element methods, in which only the boundary of the excavation is divided into elements and the interior of the rock mass is represented mathematically as an infinite continuum. The boundary element method is relatively simple, quick and cheap to run but is limited to elastic analysis. The rock mass is represented as a continuum of infinite extent, making it difficult to incorporate variable material properties and structure and to model support interaction.
- Domain methods, in which the interior of the rock mass is divided into geometrically simple elements, or zones, each with assigned properties. The collective behaviour and interaction of these simplified elements model the more complex overall behaviour of the rock mass (Hoek et al. 1995). Domain methods require the outer boundaries of the model to be placed sufficiently far from the excavations that errors arising from the interaction between these outer boundaries and the excavations are reduced to an acceptable minimum.

  'Finite element' and 'finite difference' methods are domain techniques which treat the rock mass as a continuum. They permit elastoplastic analysis and so can be used to solve for failure mechanism and to model behaviour during failure. Both are well suited to solving rock mechanics problems involving non-linear and heterogeneous material behaviour, by applying different material properties and constitutive laws to different zones.

  The 'discrete element' method is also a domain method which models each individual block of rock as a unique element so as to simulate the mechanical response of discrete blocks or particles. Discrete element models have applications where the ground is blocky and intersecting joints form rigid blocks and wedges of rock.

The advent of computing technology has resulted in numerical analysis becoming a powerful and valuable tool in ground engineering. Elements of geotechnical systems that could only be evaluated previously as discrete units can now be analysed in the context of a composite and interactive system and outcomes can be subjected to a range of parametric, sensitivity and probabilistic analysis. Complex geological and/or geometric conditions can be simulated although, depending on model scale, it can be difficult to represent geological structure adequately. Numerical models enable the state of stress and strain to be evaluated at virtually any point in the rock mass. The more advanced models offer the advantage of being able to evaluate the effect of coupled fluid flow on rock mass behaviour. Jing (2003) presents a more comprehensive review of numerical modelling techniques and their various strengths and limitations.

Pre-requisites for sensible numerical modelling are the selection of an appropriate numerical code and the calibration of the model against observed rock mass response. It is important to appreciate that model calibration on the basis of back-analysis does not always guarantee unique solutions since different constitutive laws, numerical methods and boundary conditions may yield the same result. Comparison with vertical displacement and curvature at the ground surface provides one of the most reliable means of calibration and verification, confirming that large scale stiffness and deformation properties of the overburden, including caved material, are simulated in a reasonable manner.

Modifying material input values as a means of calibration needs to be undertaken with extreme care. It can seriously affect predicted failure pathways and result in misleading and potentially dangerous outcomes if material input values are not the true cause of deviations between predicted and measured behaviour.

Numerical modelling outcomes are always critically dependent on the validity of the constitutive laws and the failure criteria assumed in the model. The global extent of a numerical model, the geological and geometric detail contained within the modelled zone and the density of points for which results have to be computed are constrained by computational power. This means that artificial boundaries have to be applied to the model, with prescribed states of stress or displacement at these boundaries. These prescribed states are known as **boundary conditions**.

Particular care is required when dealing with non-linear systems and systems that are already beyond their yield point, as rock mass response depends upon the sequence of loading. In situations where yield has already been initiated, the current state of stress may have been influenced significantly by the loading history, or stress path, to which the material was subjected. Therefore, when undertaking elasto-plastic analysis, it is important to excavate the material in accordance with the actual extraction sequence in order to generate load and model yield in an incremental and historically correct manner. Numerical models offer powerful benefits in this regard. It is also important to be aware that stress pathways can have implications for the validity and reliability of empirical databases if the data are sensitive to variations in stress paths.

Parametric analysis involves identifying the primary variables that govern behaviour. Sensitivity analysis is concerned with assessing how outcomes are affected by variation in input values for a parameter. Provided that the model reproduces field behaviour mechanisms with a reasonable degree of accuracy, numerical modelling is particularly useful for undertaking parametric studies and sensitivity analysis to assess the impact of uncertainties and inaccuracies in modelling input values. This approach can provide insight into mechanisms of deformation and aid in both interpreting field observations and measurements and in developing sensible empirical relationships. Numerical modelling may be used on a comparative basis to assess the role of critical parameters, rather than in a deterministic manner to derive absolute predictions of ground behaviour.

Knowledge, judgement and experience are required in selecting a numerical model as different models simulate different rock mass behaviour modes. It is useful, but not essential, for an end-user to have a basic understanding of the mathematical complexity of the numerical model. However, it is important to have a clear understanding of the physical concepts that the model embodies and their limitations.

The end-user should always seek clarification of the simplifying assumptions used in the construct of a model, including constitutive laws, failure criteria, boundary conditions and material properties, and the limitations associated with these. Cognisance should also be taken of the experience of Brown (2011) that, despite the vast range of knowledge and experience that is available in this field, the application of these methods in engineering practice often suffers because some analysts regard the computer codes used as “black boxes” and pay insufficient attention to the mechanics of the problems concerned, the input data and to the meaning or “believability” of the results obtained. Furthermore, there is a tendency to disregard features of a problem that are not catered for specifically in the software selected or available for use.

Bieniawski (2011) expressed the view that there has been a distinct trend for using “convenient” continuum codes, which have particularly good graphics representation of results and that, as a result of the availability of simple software packages, a user might need only limited understanding of rock mechanics principles to use the codes “successfully”. As a result, a consultant’s report might contain endless colourful stress distributions and deformation patterns but, questions Bieniawski, does this ‘colour’ mean anything real? The opinion of Brown (2012a) is endorsed that many seeking to use modern numerical methods in rock engineering design analysis should provide greater attention to the guidance provided by Starfield and Cundall (1988), especially their warning that numerical modelling is an aid to thought rather than a substitute for thinking The reader is referred to Wiles (2006) for a more in-depth discussion of the reliability of numerical modelling predictions.

### 2.7.4 Safety Factor

Safety factor is a traditional engineering approach to assessing stability by comparing capacity to demand, or strength to working stress (Eq. 2.27). The reliability of the outcome depends on the confidence that can be placed in the determination of both the resisting forces and the driving forces. The concept relies on the principle that the higher the uncertainty associated with these determinations and/or the higher the consequences of failure, the higher the design safety factor.

$$\begin{aligned} Safety\ Factor &= \frac{Capacity}{Demand} = \frac{Resisting\ Force}{Driving\ Force} \\ &= \frac{Strength}{Working\ Stress} \end{aligned} \tag{2.27}$$

One of the attractions of the safety factor concept is that it provides a simple means for communicating to non-professionals and the layman. However, there are a number of limitations associated with it that both the designer and the end-user need to be aware. In particular:

- Multiple values of safety factor can apply to circumstances involving the same level of risk. Conversely,
- The same value of safety factor can be applied to circumstances that involve widely different levels of risk.
- The design safety factor may over or underestimate the true risk associated with the design outcome, depending on the experience base of the personnel selecting the value of the safety factor.
- To the uninformed, the concept of safety factor can create the perception of 'certainty' or 'certainty plus'.

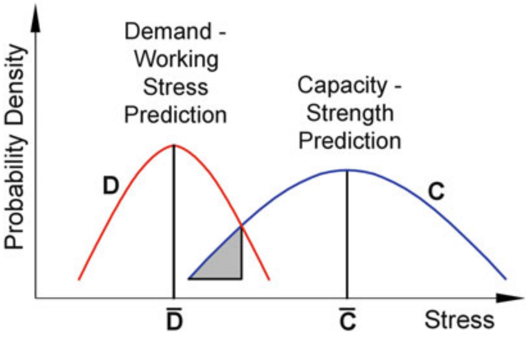


**Fig. 2.33** Probability density distribution concept relating to factor of safety

The concept of safety factor has found extensive application in ground engineering but is now superseded in many applications by adopting a stochastic approach and assigning probability density functions, as shown in Fig. 2.33, to both the demand and the capacity elements of design to reflect the uncertainties associated in their determination. The curves C and D reflect the range in predictions of capacity and demand, with $\overline{C}$ and $\overline{D}$ being, respectively, the mean capacity and the mean demand. The shaded zone encapsulates those situations where working stress exceeds strength, the area of this zone being a measure of the probability of instability.

If the design strength and the actual working load were to be known precisely, then a safety factor of more than 1 would imply stability. Conversely, a safety factor of less than 1 would imply an unstable outcome. Hence, a safety factor of 1 equates to a 50 % chance of stability and a 50 % chance of instability. This level of precision is not achievable in ground engineering and so there is a need to increase safety factor with increasing uncertainty in strength and/or working stress determinations. For this reason, a factor of safety is, effectively, a 'factor of ignorance', with only the very well informed or the foolish designing to safety factors approaching 1.

The sensible use of safety factors that are not underpinned by statistical analysis requires education, experience and judgement. The advice of the Institution of Engineers Australia should be borne in mind in this regard (IEAust 1990), being that a safety factor is a means of problem solving based on experience; that is, it is a heuristic, or a 'rule of thumb'. Every factor of safety is a heuristic because it does not guarantee an answer. Rather, it competes with other possible values and it depends on time and context for its choice. Nevertheless, it is used because it reduces the effort needed to obtain a satisfactory answer.

### 2.7.5 Statistical and Probabilistic Analysis

Uncertainty in ground engineering is pervasive across all aspect of the discipline. For example, it is associated with selecting measurement sites, measurement techniques, measured data values, formulations for processing data, gaps in knowledge, and variance between design and as-built. Provided that they are used correctly, statistics and probability theory are powerful tools for managing risk associated with these types of uncertainty in ground engineering. Conversely, they can elevate risk if they are used inappropriately or are pushed beyond their limits. The later has become more prevalent since the advent of software packages for statistical and probability analysis that are incorporated into spreadsheet programs.

This section introduces some basic statistical and probabilistic analysis techniques that find application in ground engineering and provides guidance on aspects of their use. It is not intended to be comprehensive. The reader is referred to publications such as Whitman (1984) and Harr (1997) for more detailed information.

As a starting point, it is important to appreciate the theoretical difference between a statistical approach and a probabilistic approach to managing uncertainty and how this distinction has come to be blurred in ground engineering. Mathematically, **statistics** is concerned with inferring properties about a population based on a random sample from the population. Conversely, **probability theory** is concerned with the chance of achieving an outcome from sampling a population about which everything is known. Statistical approaches enable the **likelihood** of an outcome to be predicted, as distinct from the **probability** of an outcome derived from probabilistic approaches.

In ground engineering, one is usually dealing with samples from a population, such as borehole cores, and so it might be expected that statistical approaches dominate. However, it has become established practice to treat some sample populations as being representative of the whole population and to apply probabilistic approaches to assessing the reliability of design approaches based on this sample data. This merging of the concept of likelihood and probability is becoming embedded in ground engineering, although it still gives rise to some confusion and debate.

In keeping with many other references in ground engineering, probability theory is not distinguished from statistical analysis in this text. However, the reader needs to be aware of the risks associated with this approach. The reliability of the probabilistic predictions depend on how representative the sample bases are of the whole population and, therefore, are not fully commensurate with a rigorous probabilistic approach to risk analysis. Rather, they continue to represent the likelihood of an outcome based on a sample of a population. This risk is particularly apparent in some linear regression and logistic regression approaches to formulating mine design procedures and the confidence levels that proponents associate with these procedures.

According to Brown (2012b), probabilistic risk analysis is probably the most widely-used approach to risk assessment in rock engineering and in geotechnical engineering more broadly. Notwithstanding this, the methodology has found limited application to ground engineering in underground coal mining, albeit that a range of design approaches in this sector are premised to some degree on statistical analysis.

The simplest statistical analysis measures are the **median**, the **arithmetic mean**, the **variance**, and the **standard deviation**. The medium score for a set of observations is that value which divides the database into two intervals having equal frequency. That is, 50 % of cases fall above the median and 50 % fall below it. The median score is the position occupied by the median in a sequence of n data values placed in numerical order. It is calculated using either Eq. 2.28 or 2.29, depending on whether the number of points in database is an odd or even number.

$$Median\ score = Md = \frac{(n+1)}{2} \tag{2.28}$$

where

$n =$ *an odd number of data values*

$$Median\ score = Md = \frac{n}{2} \tag{2.29}$$

where

$n =$ *an even number of data points*

The arithmetic mean is simply the average of the values making up the database. It is calculated using Eq. 2.30.

$$Mean = \bar{x} = \frac{1}{n}\sum_{i=1}^{n} x_i = \sum_{i=1}^{n} \frac{x_i}{n} \tag{2.30}$$

where

$x_i =$ *the* $i^{th}$ *data value*

The level of confidence that can be placed in an outcome falling close to the arithmetic mean is dependent on how closely the database is spread or dispersed about the mean value. The variance and the standard deviation provide measures of this degree of variability. Variance, $s^2$, is calculated by averaging the sum of the square of each deviation from the mean (Eq. 2.31). One drawback with variance is that the outcome no longer has the same dimensional units as the data points to which it relates, since the dimensions of the data points are squared. This is easily overcome by taking the square root of the variance, to produce the standard deviation, s, given by Eq. 2.32.

$$Variance = s^2 = \frac{1}{n}\sum_{i=1}^{n} (x_i - \bar{x})^2 \tag{2.31}$$

$$Standard\ Deviation = s = \sqrt{\frac{1}{n}\sum_{i=1}^{n} (x_i - \bar{x})^2} \tag{2.32}$$

**Linear regression** is one of the most widely used statistical techniques. It is concerned with establishing a level of confidence that there is a relationship between a determinant (or independent variable), x, and an outcome (or dependent variable), y. The process is referred to as **simple linear** regression when only one determinant variable is involved and **multiple linear regression** when dealing with more than one determinant variable. It involves fitting a line to the data using the process of least squares (which minimises the average of the square of the errors in predictions). The **goodness-of-fit** of this line or relationship is defined by the **coefficient of determination**, $r^2$, where r is the **correlation coefficient** given by Eq. 2.33 for the case of a sample of bivariate data pairs (x, y).

$$Correlation\ Coefficient = r = \frac{\sum (x_i - \bar{x})(y_i - \bar{y})}{\sqrt{\sum (x_i - \bar{x})^2 \sum (y_i - \bar{y})^2}} \tag{2.33}$$

where

$\bar{x} =$ *mean x value*
$\bar{y} =$ *mean y value*

Linear regression analysis is useful to visualise and plot curves that best describe the range and behaviour of a data set. It is often used, for example, to evaluate the degree to which Young's Modulus correlates to uniaxial compressive strength and, therefore, to assess the reliability associated with specifying Young's Modulus based on uniaxial compressive strength. However, in many other cases in ground engineering, risk may be associated with this simple approach because it can circumvent the development of a proper understanding of the mechanics of the problem.

Unlike in probabilistic based approaches, one cannot specify what constitutes an acceptable

**Table 2.8** Guidance to assigning significance to the coefficient of determination, $r^2$, as provided by McNeill (2001)

| $r^2$ | Goodness-of-fit |
|---|---|
| 0–0.3 | Poor |
| >0.3–0.7 | Moderately good |
| >0.7–0.9 | Very good |
| >0.9 | Excellent |

value for goodness-of-fit in linear regression as it depends on the situation. McNeill (2001) advised that if a verbal summary had to be provided, he would adopt the values summarised in Table 2.8. It should be borne in mind that the relationship defined by the line of best fit may not be mechanistically valid or complete. It is also important to appreciate that while high standard deviations are associated with low $r^2$ values, high standard deviations can also be associated with seemingly good $r^2$ values. These are important considerations when deciding whether to adopt designs and analyses that rely to some extent on linear regression procedures.

**Logistic regression** utilises the statistical method known as **maximum likelihood** in an endeavour to determine values for parameters that influence an outcome such that a binary outcome falls distinctly into one of two categories, labelled 'positive' or 'negative'. These categories can be designated, for example, as 'successful' and 'unsuccessful'. The probability of a successful or unsuccessful outcome can be calculated using logistic regression. The methodology is based on the linear relationship given by Eq. 2.34. It is concerned with determining which linear combination of variables, $x_i$, has the most influence on determining an outcome and what regression coefficient, $B_i$, for each variable maximises the likelihood of successfully predicting that outcome.

$$g(x) = B_0 + B_1x_1 + B_2x_2 + \ldots + B_nx_n \quad (2.34)$$

where

$x_i = a\ variable$
$B_i = a\ regression\ coefficient$

The more positive the value of g(x), the greater the likelihood of a positive outcome and, conversely, the more negative the value, the greater the likelihood of a negative outcome. Hence, if $g(x) = 0$, the likelihood of an outcome is 0.5, or 50 %. The analysis can be extended by expressing g(x) as a mathematical logit function to produce suites of equations that correspond to given confidence levels of avoiding an unsuccessful outcome.

A limitation with logistic regression is that the process can only deal with linear relationships between a variable and g(x). Therefore, other relationships (e.g. quadratic, logarithmic) have to be able to be transformed into a linear form while retaining a sensible physical relationship. This is not always achievable, or only to a limited extent. Because the methodology is not premised on a mechanistic approach, care is required that some influences are not 'double counted' as a result of two (or more) variables having a close correlation, and that variables eliminated from a model because they have little effect on the outcome do not become significant later when a different combination of variables, or data for the variables, become available.

The methodology defines the probability of correctly identifying a positive outcome as **sensitivity** and the probability of correctly identifying a negative outcome as **specificity**. Figure 2.34 is a demonstration example of sensitivity and specificity plotted against the probability of predicting the correct associated outcome. The optimum 'cut-off' point occurs at the cross over point between the sensitivity and specificity curves, as this maximises both sensitivity and specificity, corresponding to the likelihood of detecting a false positive equalling the likelihood of detecting a false negative. The technique suffers from the same limitation as linear regression in that there is no universally accepted measure of goodness-of-fit that can be translated into a quantifiable risk outcome.

One approach in these circumstances is based on the area under the so-called **ROC curve** (or 'receiver operating characteristic' curve) produced by plotting sensitivity against [1-specificity]. The ROC curve corresponding to the demonstration data plotted in Fig. 2.34 is shown in Fig. 2.35. Table 2.9 records the significance which Hosmer and Lemeshow (2000)

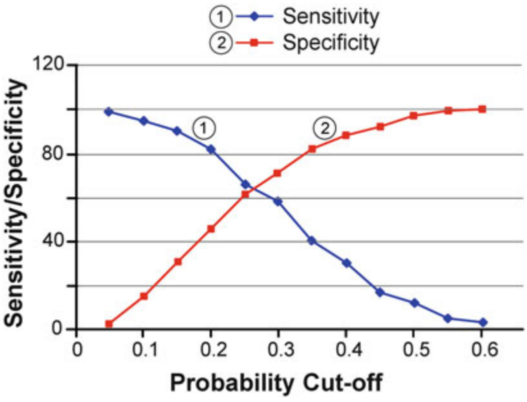


**Fig. 2.34** A demonstration example of sensitivity and specificity plotted against probability cut-off

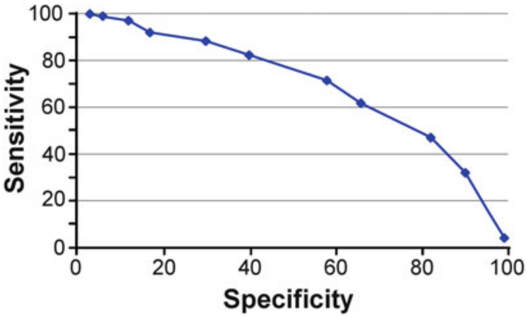


**Fig. 2.35** ROC curve showing sensitivity plotted against [1-specificity] for the demonstration data used to plot Fig. 2.34

advise, as a general rule, can be applied to ROC area calculations.

The deficiencies in safety factor, logistic regression and linear regression for quantifying probability in risk management can be addressed by adopting a **stochastic** approach. This approach is particularly powerful for managing uncertainty in design procedures that have a reliance on input parameters that are **random variables**. That is, they have no fixed value. Examples of random variables include uniaxial compressive strength, Young's modulus and friction angle.

The relative likelihood that a random variable will assume a particular value can be described by a probability distribution curve, or **probability distribution function** (PDF), of a form shown in the demonstration example in Fig. 2.36a. In the case of Curve A, the outcomes are distributed symmetrically about a value of 5.5, while for curve B they are skewed to the higher end. Curve A could conceivable represent the outcomes of a uniaxial compressive testing program, while curve B might represent the outcomes of a survey of roadway width.

**Table 2.9** Guidance to assigning significance to ROC curve for logistic regression outcomes provided by Hosmer and Lemeshow (2000)

| Area under ROC curve | Discrimination |
|---|---|
| 0.5 | None |
| 0.7–<0.8 | Acceptable |
| 0.8–<0.9 | Excellent |
| >0.9 | Outstanding |

The same information can be presented in the form of a cumulative plot, or **cumulative distribution function** (CDF), shown in Fig. 2.36b. This gives the probability that a random variable will have a value less than or equal to a selected value. In the case of curve A for example, there is a 50 % probability that a sample will return a uniaxial compressive strength less than or equal to a value of 5.5 (and similarly, a 50 % probability that it will return a value greater than 5.5). Curve B shows that there is only a 40 % probability that measured roadway width will not exceed 5.5 m (or conversely, a 60 % probability that roadway width will exceed 5.5 m).

The area under a PDF curve is always equal to unity (1) because it represents 100 % of the outcomes. The CDF curve shows the manner in which the area under the PDF curve increases with distance along the abscissa, or X axis. That is, the CDF curve is the integral of the PDF curve. The mean value, $\bar{x}$, of a set of variables is sometimes referred to as the **first moment**, or **centre of gravity** of the probability distribution and the variance, $s^2$, as the **second moment**.

The shape of a PDF curve can be defined by various mathematical functions, or distributions based on 'goodness-of-fit' analysis, with the most commonly employed distribution being a normal or Gaussian distribution. Other more frequently employed continuous distributions in ground engineering include lognormal, Beta,

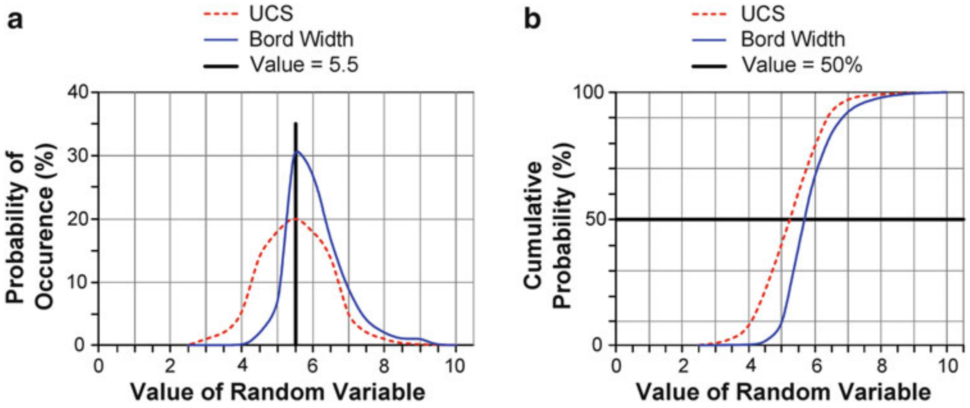


**Fig. 2.36** Demonstration examples of probability distribution function (PDF) curves and their corresponding cumulative distribution function (CDF) curves. (**a**) Probability distribution function curves (**b**) Cumulative probability distribution curves for those shown in (**a**)

Gamma, and Weibull. The PDF distributions form the basis for undertaking Monte Carlo simulation, which involves selecting an appropriate model that produces a deterministic solution to a problem, randomly sampling input values from a suite of PDF curves, and computing the likelihood of a specified outcome for this combination of values. The random sampling process is repeated many thousands or hundreds of thousands of times to generate an overall probability distribution profile of a specific outcome.

In some distributions, including a normal distribution, variables may fall in the range of $-\infty \leq x \leq \infty$. This can lead to problems when undertaking Monte Carlo analysis based on randomly sampling values across their full range and so the distribution is sometimes truncated to remove values falling at the extremity of the sampling range. This problem may also be overcome in many instances by using alternative PDF distributions that are non-negative.

Monte Carlo analysis is well suited to evaluating risk arising from uncertainty in input values when the problem being analysed is defined by sets of equations. However, it is of limited value in situations where load and, therefore, stress are indeterminate. Nevertheless, as noted by Whitman (1984), while it is not possible to utilise probabilistic techniques such as the Monte Carlo analysis directly in the analysis of stress driven instability, it is useful to consider the possible range of input parameters when working with these problems. Hence, when using a numerical model to analyse the extent of the failed zone around an opening, for example, it is wise to run the model several times to investigate the influence of variations in applied stresses, rock mass properties and the characteristics of different support systems.

An historically contentious probabilistic assessment approach which is gaining increased attention and application in ground engineering is that of Bayesian theory. This theory dates back to the seventeenth century and has been found to be highly successful when dealing with low probability, high consequence situations, for which there is usually a severe lack of objective data. In simple terms, Bayesian theory revolves around forming an initial, or prior, hypothesis on the basis of limited data and updating and improving this hypothesis in the light of the likelihood that new objective data may or may not support the hypothesis. The improved hypothesis becomes the prior hypothesis with each iteration.

The process is defined by Bayes' rule, given in Eq. 2.35.

$$p(H|E) = \frac{p(E|H)p(H)}{p(E|H)p(H) + p(E|\overline{H})p(\overline{H})} \quad (2.35)$$

where

$p(H|E)$ = *the probability of a hypothesis being correct given additional evidence*

$p(H)$ = *the prior probability that a hypothesis is correct*

$p(\overline{H})$ = *the prior probability that a hypothesis is incorrect*

$P(E|H)$ = *the probability of observing in the additional evidence, evidence that supports the hypothesis when it is correct*

$P(E|\overline{H})$ = *the probability of observing in the additional evidence, evidence that supports the hypothesis when it is, in fact, incorrect*

According to Whitman (1984), Bayesian updating (or reliability updating) may be viewed as a formalisation of the observational approach advocated by Terzaghi, Peck and others. The methodology is very sensitive to the acquisition of new data, with one new fact having the potential to significantly alter the probability of a situation. The heart of the controversy surrounding the method is that when only limited data is available, the outcome of a Bayesian computation depends on prior opinion. In such a case, Bayes' rule can lead to a subjective rather than an objective assessment of a situation (McGrayne 2011).

Whitman (1984), Vick (2002) and Christian (2004) provide more in-depth discussion on probabilistic approaches that find application to quantifying and managing uncertainty and reliability in ground engineering A detailed account of the development and application of Bayes' rule is provided by McGrayne (2011).

## 2.8 Statics

### 2.8.1 Introduction

In Sect. 2.2.1, it was noted that the structural and mechanical characteristics of an underground coal mining environment result in beam, column and plate theory finding application to ground engineering and mine design. This requires a basic understanding of statics, dynamics and kinematics, which are branches of mechanics concerned with how a body responds to forces, displacements and constraint. Classical beam theory, in particular, is a very useful element of statics for conceptualising the behaviour of strata beds and developing an appreciation of which parameters have the most impact on strata response to mining.

In this section, the behaviour modes of beams, columns and plates are considered based on the ideal conditions on which classical beam theory is premised. In practice, the situation is usually more complex due to a host of interacting factors that do not all satisfy these ideal conditions or the scope of the theory. Nevertheless, classical beam theory is still invaluable in conceptualising and evaluating structural behaviour in both underground and desktop settings and in making informed appraisals and decisions about the merits, limitations and confidence that can be placed in ground support design procedures and stability assessments based on the theory.

### 2.8.2 Basic Definitions and Principles

**Statics** involves the analysis of forces and moments on bodies that are in **static equilibrium**. A body or system is in a state of static equilibrium when it is either at rest or moving at a constant velocity. In accordance with Newton's laws of motion, this requires that both the sum of all the forces acting on the system is zero and the sum of all the **turning moments** (or lever arms) acting on the system is zero (a turning moment being equal to 'force x distance to point of application of the force'). Diagrams which depict the relative magnitude and direction of all forces acting on a body are referred to as **free body diagrams**.

**Dynamics** is concerned with the motion of bodies under the actions of forces and moments. **Kinematics** addresses the geometrically possible motions of a body without regard to the forces and moments that generate the motions.

In engineering mechanics, **bending** is referred to as **flexure**. Structural members which offer resistance to flexure when subjected to load are

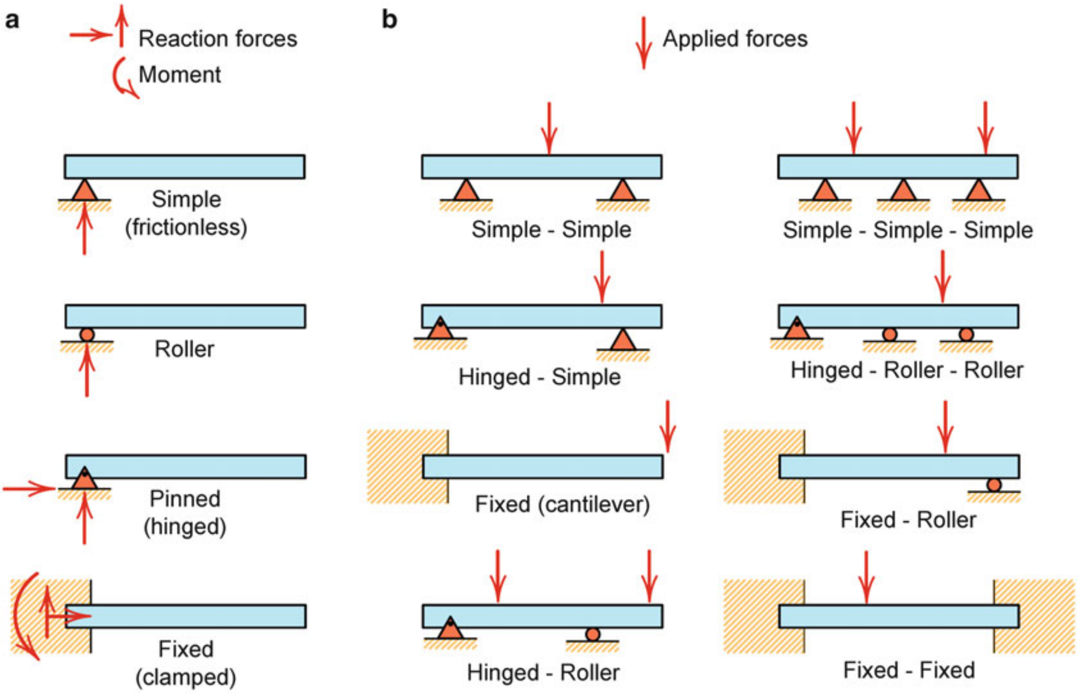


**Fig. 2.37** Types of beam supports and loading situations. (**a**) Support methods (**b**) Loading situations

referred to generically as beams and include columns and plates. More specifically, a **beam** is a structural member that is loaded transversely and a **column** is a structural member that is loaded longitudinally in compression. A **plate** is a beam that has a width of the same order of magnitude as its length. A structural member that is loaded simultaneously in compression in the transverse and longitudinal directions may be referred to as a **beam-column**. Strata in an underground coal mine can be loaded in all of these manners.

The means by which a beam is supported has a significant influence on its behaviour under load. Three common support methods, or connections, are **roller**, **pinned**, and **fixed**, shown in Fig. 2.37. A roller support is capable of resisting only one component of force, being that normal to the surface on which the roller acts. The support is free to translate along the surface on which it rests and cannot provide resistance to lateral forces. A pinned support can resist both vertical and horizontal forces but not a turning moment, or **couple**. It allows the beam to rotate but not to translate in any direction. A fixed support can resist vertical and horizontal forces as well as a turning moment. Hence, it does not allow a change in slope at the support point. The clamped end of a beam constitutes a fixed support.

A beam which cannot undergo any translational displacement at its support points but is free to rotate about them is referred to as a **simply supported beam**. If a beam is only supported at one end it is referred to as a **cantilevered beam**. A beam supported in a manner which allows its reactions to load to be calculated by statics alone is classified as **statically determinate**. When a beam has more supports than necessary to achieve a state of equilibrium, the load reactions cannot be determined by simply balancing the forces and moments. This type of situation is said to be **statically indeterminate** and requires consideration of the stiffness properties of the beam in order to resolve load distributions and reactions. Figure 2.37 shows examples of these various support and loading situations.

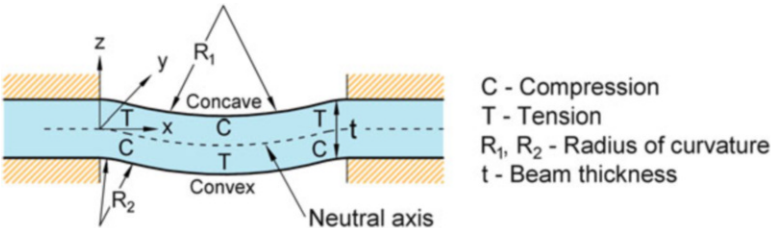


**Fig. 2.38** Features associated with flexure of a beam

**Euler-Bernoulli theory**, or so-called **classical beam theory**, invokes a number of ideal assumptions that enable beam loads and displacements to be calculated using a simplified form of linear elastic theory. These assumptions include that the beam is:

- isotropic, homogenous, and free of defects;
- linearly elastic and will not deform plastically;
- perfectly straight along its axes and of constant cross-section when in an unloaded state;
- of uniform flexural rigidity;
- initially stress free;
- loaded only normal to its faces;
- symmetric about an axis in the plane of bending.

In any given interval along a beam subjected to bending, one surface will be stretched and the opposite surface will be compressed. These effects decrease toward the centre of the beam cross-section until, as shown in Fig. 2.38, a neutral point is reached that experiences neither tension or compression. The loci of these points constitutes the **neutral axis**, which does not change in length under the effects of bending. The radius of the bending profile is known as the **radius of curvature,** R, and is a measure of the severity of bending as shown by the radii $R_1$ and $R_2$ in Fig. 2.38. A surface which bends out on itself is referred to as **convex** and is associated with **hogging** and **tension**. A surface which bends in on itself is referred to as **concave** and is associated with **sagging** and **compression**.

Provided that a beam is no thicker than one-fifth of its length, its weight can be approximated by applying a **uniformly distributed load**, q, per unit width to its top surface. This load is given by Eq. 2.36:

$$q = \rho g t = \gamma t \quad (N) \tag{2.36}$$

where

$\gamma = unit\ weight\ of\ the\ beam$
$t = beam\ thickness$

The **second moment of inertia**, I, is a measure of the distribution of the cross-sectional area relative to the neutral axis of a beam and defines the influence of the cross-sectional profile of a beam on its capacity to bend under load. In ground engineering, most analysis is based on beams with a rectangular cross-section, for which the second moment of inertia is given by Eq. 2.37.

$$I = \frac{bt^3}{12} \quad \left(m^4\right) \tag{2.37}$$

where

$b = beam\ width\ (usually\ taken\ to\ be\ unit\ width\ )$

The relationship between the transverse load acting on a beam and **beam deflection**, δ, is referred to as **bending stiffness** or **flexural rigidity**, defined by Eq. 2.38.

$$Flexural\ Rigidity\ = EI \quad (N) \tag{2.38}$$

Flexural rigidity determines how load is transferred between stacked beams or plates. A beam with a lower flexural rigidity than an overlying beam will deflect more under the same load, thus creating a void between the beams. Conversely, a beam with a lower flexural rigidity than an underlying beam will transfer load to the lower beam.

### 2.8.3 Transversely Loaded Beams

Static evaluation is concerned with both the external and internal effects of load on a beam. Evaluation of the external effects requires the resolution of forces and moments so that the net resultant of each is zero and equilibrium is maintained. This concept is illustrated in Fig. 2.39.

Evaluation of the internal effects of load is concerned with the behaviour of the beam fabric under load. The first factor to consider is the capacity of the beam to support its own weight and any additional surcharge (transverse) load and to transfer the total load to the beam supports (Fig. 2.40). The load generates shear forces, V, in the transverse direction, promoting failure of the beam by sliding on planes in that direction. In an end supported beam, shear force is zero in the transverse plane at the mid-span of the beam and increases towards the abutments in direct proportion to beam length, as depicted in Fig. 2.41. The average shear stress, $\tau_{xz\ ave}$, in any plane of area, A, normal to the neutral axis in the beam is given by:

$$\tau_{xz\ ave} = \frac{V}{A} \tag{2.39}$$

where

$\tau_{xz\ ave}$ = *average stress acting in z direction on the yz plane*
$V$ = *transverse shear force acting on the yz plane*

The second internal factor to consider is axial stress distribution and magnitude within the beam. Beam curvature generates axial (or longitudinal) tensile and compressive forces either side of the neutral axis. These forces, in turn, generate a bending moment couple in the beam as shown in Fig. 2.40. For small deflections, the **internal bending moment**, M, is given by Eq. 2.40.

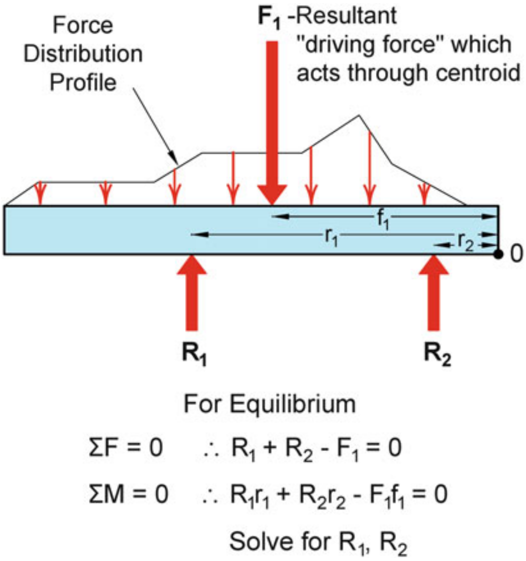

Force Distribution Profile
$F_1$ -Resultant "driving force" which acts through centroid
$f_1$
$r_1$
$r_2$
0
$R_1$
$R_2$
For Equilibrium
$\Sigma F = 0 \quad \therefore R_1 + R_2 - F_1 = 0$
$\Sigma M = 0 \quad \therefore R_1r_1 + R_2r_2 - F_1f_1 = 0$
Solve for $R_1$, $R_2$

**Fig. 2.39** Overview of the process to achieve static equilibrium in a transversely loaded beam

$$M = \frac{EI}{R} = EI\frac{d^2z}{dx^2} = \frac{\mathrm{E}}{\mathrm{R}}\iint \mathrm{z}^2\mathrm{dx.dz} \tag{2.40}$$

where

$M$ = *bending moment at section in question*
$R$ = *radius of curvature at section in question*

The axial stress induced in a beam is referred to as **fibre stress**, $\sigma_f$. The magnitude of this stress at any point, p, in the beam (Fig. 2.40) is given by Eq. 2.41.

$$\sigma_f = \frac{Mz}{I} \tag{2.41}$$

where

$M$ = *bending moment at point p*
$z$ = *the normal distance from the neutral axis to point p*

The maximum fibre stress that can be sustained in a beam is defined as the **flexural strength** and occurs at the outer extremities of

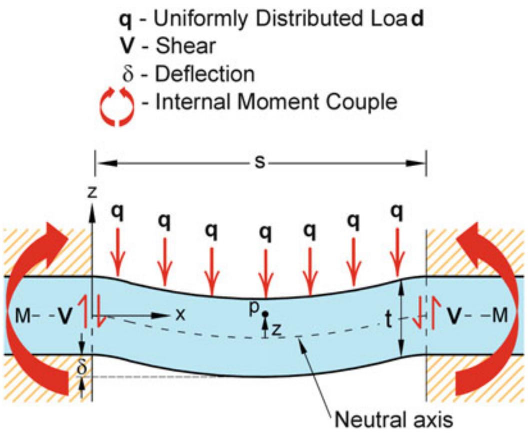


**Fig. 2.40** Distribution of deflection, tension, compression and shear in a uniformly loaded beam with clamped ends

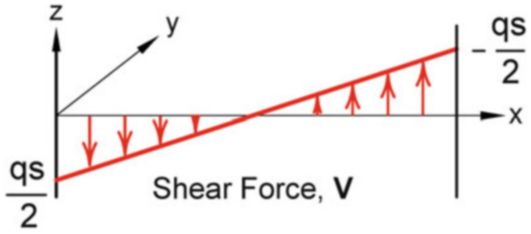


**Fig. 2.41** Shear force diagram for a transversely loaded beam with either clamped ends or simply supported ends

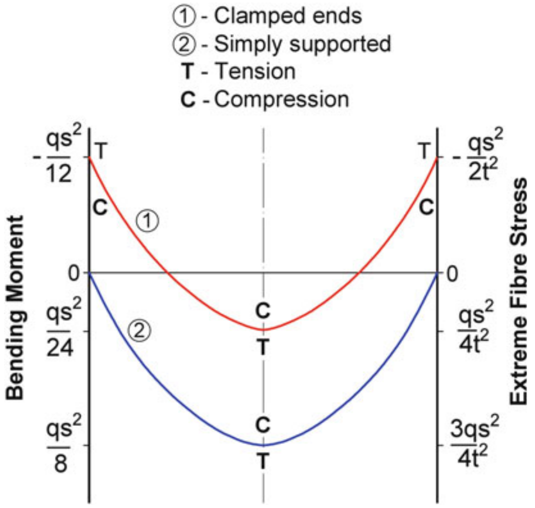


**Fig. 2.42** Bending moment diagrams for a beam of unit width with clamped ends and for a beam of unit width with simply supported ends

the beam cross-section. The locations of the peak tensile and compressive stress on these outer surfaces depend on the manner in which the beam is supported. Figure 2.42 shows that in the case of a simply supported beam, the peak tensile stress occurs on the lower surface at mid-span, while for a beam with clamped (fixed) edges, it occurs on the upper surface over each abutment. This figure illustrates how analysis outcomes are influenced significantly by assumptions regarding beam support conditions.

The generation of axial tensile and compressive stresses in a beam sets up a second set of shear stresses, which are designated $\tau_{zx}$ (because they act normal to the xy plane and are orientated in the x direction). Figure 2.43 shows how **axial shear stress**, $\tau_{zx}$, is zero at the top and bottom surfaces of the beam, since these are free surfaces, and a maximum along the neutral axis, since the beam fibres are in tension on one side of this axis and in compression on the other side. The shear stress profile on a transverse plane is a function of the shape of the beam. For a beam with a rectangular cross section, the maximum shear stress occurs at the neutral axis and is 1.5 times the average shear stress, V/A, on that plane (Fig. 2.43).

Shear forces also produce moments which must be balanced to maintain equilibrium. In the case of a beam, a balanced moment couple requires that $\tau_{xz} = \tau_{zx}$. Hence, the shear stress distribution shown in Fig. 2.43 applies to both transverse shear, that promotes sliding at the beam abutments, and to longitudinal shear, that promotes sliding between any laminations within the beam.

Shear force, bending moment and deflection distributions for different types of supported beam under transverse load can be calculated from first principles. However, numerous formulations are available in specialist texts such as Young et al. (2012). Suites of basic statics formulations for a clamped beam and a simply supported beam under transverse load are provided in Appendix 3. This appendix also

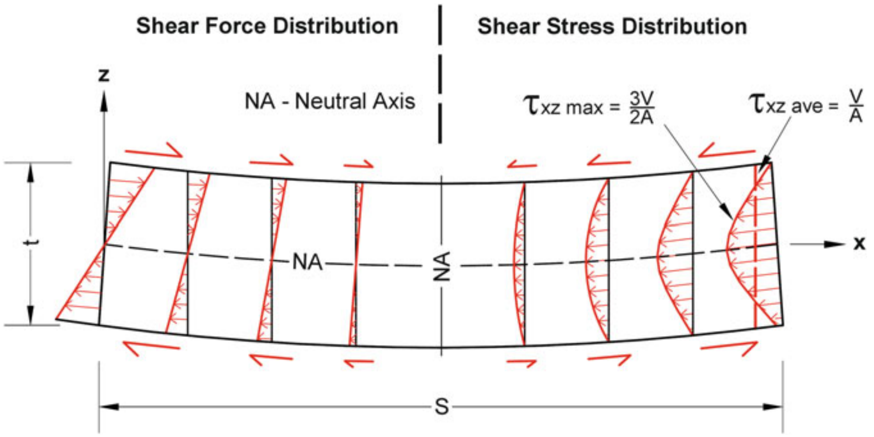

Shear Force Distribution
Shear Stress Distribution
NA - Neutral Axis
z
t
NA
NA
x
S
$\tau_{xz\,max} = \frac{3V}{2A}$
$\tau_{xz\,ave} = \frac{V}{A}$

**Fig. 2.43** Shear force and stress distribution within a deflected beam

includes the formulation for calculating load transfer and beam deflection when a beam is loaded transversely by a less stiff overlying beam.

## 2.8.4 Axially Loaded Columns

When the load in a column acts through the centre of gravity of the column cross-section, it is referred to as **axial load**. Otherwise, a load which has its line of action at any other point in the cross-section constitutes an **eccentric load**.

Depending on a column's end conditions, slenderness and straightness, and the eccentricity of the load applied to it, the compressive loading of a column may cause it to bow, or deflect, thus generating flexural stress. Deflection of a column is referred to as **buckling**, in comparison to **bending** in the case of deflection of a beam.

A column with pinned ends has no resistance to rotational moments and, therefore, will deflect more than a column with fixed ends under the same loading conditions. As a matter of convenience, it is usual to take the effect of end constraints into account in column analysis by replacing column length in base formulations with **effective length**, where effective length is the distance between successive points of zero bending moment, as shown in Fig. 2.44. In this approach, effective length is given by Eq. 2.42.

$$Effective\ Length = L_e = KL \tag{2.42}$$

where

$K = effective\ length\ coefficient$
$= 1\ for\ both\ ends\ pinned$
$= 0.5\ for\ both\ ends\ fixed\ (clamped)$
$= 2.0$ *for one end fixed and the other free to move laterally*

According to Brown (2014), the one end condition more likely than any other to be applicable to coal mine roadways is where the ends of a roof beam rotate through a small angle, φ, and transmit a moment, M. The mechanical analogue to this condition is a coiled spring that represents an otherwise pinned end, shown in Fig. 2.44e. The effective length coefficient, K, is a non-linear function of the product of the ratio of M/φ and several other terms defining the beam geometry and its elastic properties that are given in Shanley (1967). In most cases of practical interest, values of K of about 0.6–0.8 can be expected to apply (Brown 2014).

Slenderness is described by the **slenderness ratio** where:

$$Slenderness\ ratio = \lambda = L/r \tag{2.43}$$

and

$$r = the\ least\ radius\ of\ gyration = \sqrt{I/A}$$

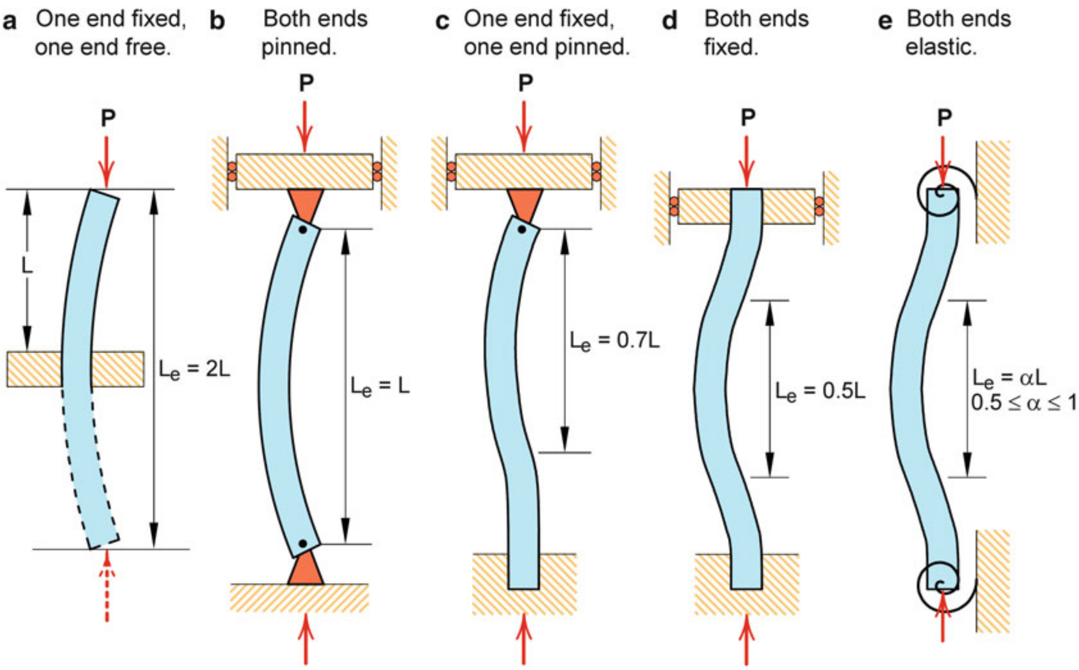


**Fig. 2.44** Influence of end constraint on the effective length of a column

For a given set of end conditions, slenderness ratio affords a means for classifying columns as either short, intermediate or long. A short column is one that undergoes compression without deflecting laterally, that is, without buckling. Increasing the axial load in a short column ultimately leads to the onset of yield due to the elastic limit of the material being exceeded, depicted in Fig. 2.45 by the segments A-B′ for theoretically ideal conditions and A-B for realistic field conditions.

A feature associated with the axial loading of a long column is that prior to exceeding the elastic limit of the material, a point can be reached beyond which load can be sustained in one of two states of equilibrium, being either stable equilibrium or unstable equilibrium as conceptualised in Fig. 2.28. Stable equilibrium is associated with elastic compression of the column in which energy has to be put into the system in order to continue to drive the deformation process. Unstable equilibrium relates to the situation where the slightest permutation will cause the system to suddenly become unstable and release energy, with the resulting dynamic failure mode being referred to in beam theory as **elastic instability**, or **Euler buckling**.

The potential for elastic instability arises when the axial load is sufficiently high that, should any eccentricity in loading develop, it will generate a turning moment sufficient to cause the column to deflect laterally. As any lateral deflection increases the turning moment, the process becomes self perpetuating, resulting in sudden failure of the column due to excessive lateral deformation. This corresponds to the segment B′-D in Fig. 2.45. Factors that can give rise to eccentric loading in what is intended to be a pure axial loading situation include defects in the internal structure of the column, the development of microfractures with increase in load, and vibration.

The maximum load that a column can maintain, theoretically, prior to becoming susceptible to buckling is referred to as the **critical load**, $P_{cr}$, or **Euler load**. It can be calculated by considering the effect of introducing a minute lateral

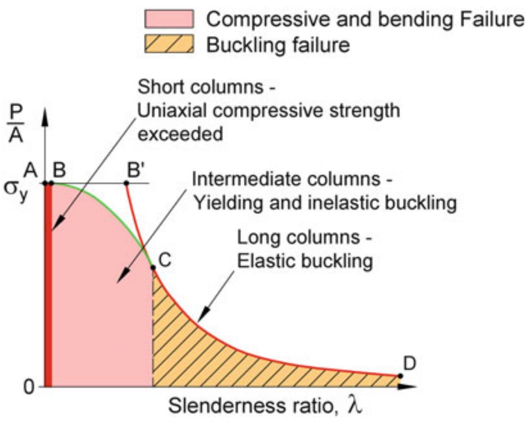


**Fig. 2.45** An example of critical unit load lines derived for short, intermediate and long columns

deviation, z′, in a perfectly straight column subjected to an axial load, P, as shown in Fig. 2.46. This deviation generates an external bending moment equal to P.z′, with stability then depending on the capacity of the column to balance this external moment with its internal moment. While ever this condition of balance is maintained at stresses lower than the elastic limit of the material, the system is said be in a state of **elastic stability**. Hence, the critical load can be determined mathematically by equating the sum of the external bending moments, $M_{ext}$, to the sum of the internal bending moments, $M_{int}$, such that the sum of all moments is zero. The steps in this process are given in Eqs. 2.44, 2.45, and 2.46. Solving Eq. 2.46 produces the Euler formula for critical load, defined by Eq. 2.47.

$$\sum M_{int} + \sum M_{ext} = 0 \tag{2.44}$$

$$\therefore M_{int} + P_{cr}.z' = EI\frac{d^2z}{dx^2} + P_{cr}.z' = 0 \tag{2.45}$$

$$\therefore EI\frac{d^2z}{dx^2} = -P_{cr}.z' \tag{2.46}$$

Solving for $P_{cr}$ gives:

$$P_{cr} = \frac{n^2\pi^2 EI}{(KL)^2} \tag{2.47}$$

where

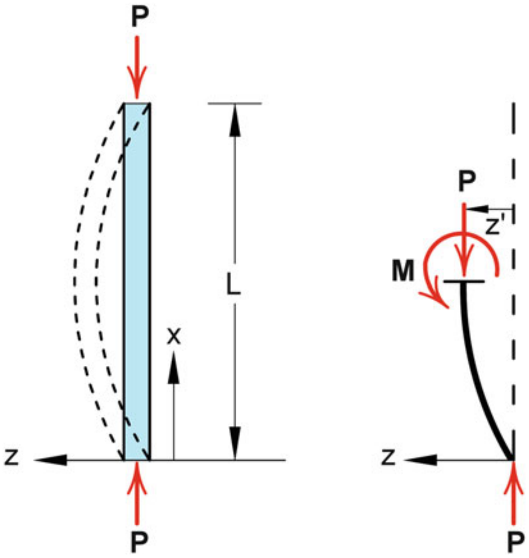


**Fig. 2.46** A free body diagram for a column under axial load with an induced lateral deflection

$n =$ *the number of possible elastic instability modes*

Although, in theory, deflection can take a number of sinusoidal forms as shown in Fig. 2.47, in practice it is usually confined to the mode corresponding to n = 1 as this requires the least driving force. Hence, the general expression for critical load in a column is:

$$P_{cr} = \frac{\pi^2 EI}{(KL)^2} \tag{2.48}$$

Equation 2.48 shows that elastic instability is not controlled by the strength of the column but rather by its flexural stiffness, EI, its length and the nature of its end constraints. For example, the critical load to cause inelastic instability of a column with perfectly rigid end constraints is four times that required for a column which has no resistance to rotation at its ends.

The critical load can also be expressed as a critical load unit by dividing it by the cross-sectional area of the column to produce the various relationships shown in Eq. 2.49. Although this expression has the units of stress and is often referred to as the **Euler stress** or **critical stress**, theoretically it is not a stress. Rather, $P_{cr}/A$ is a measure of the units of load required to generate a moment sufficient to cause buckling of a column with a given elastic modulus and geometry.

$$\frac{P_{cr}}{A} = \frac{\pi^2 EI}{A(KL)^2} = \frac{\pi^2 Er^2 A}{A(KL)^2} = \frac{\pi^2 E}{(K\lambda)^2} = \frac{\pi^2 Er^2}{(L_e)^2} \tag{2.49}$$

If the yield strength, $\sigma_y$, of the material comprising a column is known, the slenderness ratio at which Euler buckling becomes a possibility is given by Eq. 2.50, produced by rearranging Eq. 2.49 and setting $P_{cr} = \sigma_y$. Theoretically, columns with a slenderness ratio equal to or exceeding this value are classified as long columns.

$$\frac{L_e}{r} \geq \sqrt{\frac{\pi^2 E}{\sigma_y}} \tag{2.50}$$

In reality, ideal columns do not exist. Therefore, appropriate safety factors, usually greater than 2, need to be applied to Euler load predictions. Columns of intermediate length, depicted by curve B-C in Fig. 2.45, fail by a combination of direct compressive stress and flexural stress at loads less than that predicted by the Euler formula. Stability analysis of axially loaded intermediate length columns relies on a range of empirical formulae, all of which embody the slenderness ratio. Many are material specific, having been calibrated on the basis of laboratory testing. Some of the better known include Johnson's formula, the Perry Robinson formula and the Rankine Gordon formula, details of which are to found in specialist statics texts. Although some of these empirically derived formulations have been applied in ground engineering, this use is highly problematic since the formulations are not based on testing rock and do not account for the low tensile flexural strength of rock (discussed further in Sect. 7.3).

## 2.8.5 Eccentrically Loaded Columns

Fixed ended columns are not subjected to eccentric load since, by definition, the load is distributed equally over the rigid ends. The effect of the eccentric loading is simply to increase the bending moments at the ends. In other column types, an eccentric load, P, induces lateral deflection at all values of P. Therefore, the beam fabric is subjected to both axial compressive stress, P/A, and to compressive and tensile flexure (bending) stresses. Eccentrically loaded columns have no critical load and always fail as a result of bending stresses exceeding the yield strength of the material comprising the column. Eccentric loading situations are analysed by relocating the load in the manner shown in Fig. 2.48 so that it acts axially and introduces an equivalent amount of pre-existing eccentricity, e, in the column. Hence, the equation of equilibrium is now given by Eq. 2.51.

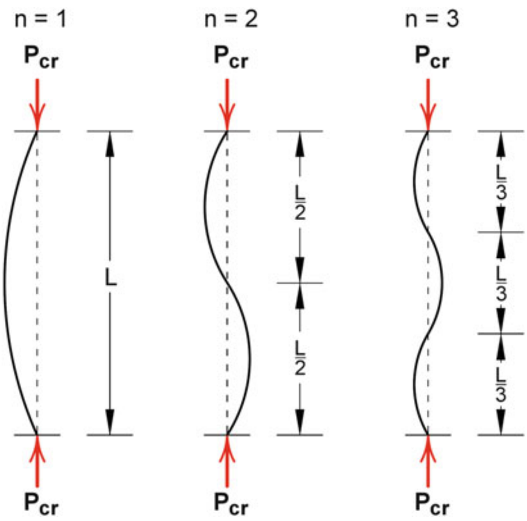


**Fig. 2.47** First three modes of elastic instability in a column

$$\sum M_{int} + \sum M_{ext} = EI\frac{d^2z}{dx^2} + P\left(z' + e\right) = 0 \tag{2.51}$$

Solving Eq. 2.51 produces Eq. 2.52 for the maximum column deflection, $\delta_{max}$. Figure 2.49 illustrates how deflection initially develops gradually and then increases at an exponential rate with increase in load, P. The eccentric load will result in an increase in compressive fibre stress

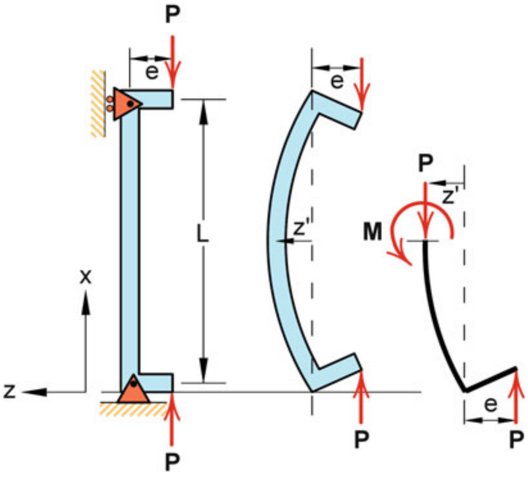


**Fig. 2.48** Column deflection due to eccentric loading

on that side of the neutral axis to which the load is biased, and a reduction in compressive axial stress on the opposite side of neutral axis. The maximum compressive fibre stresses in the column is given by Eq. 2.53.

$$\delta_{max} = e\left(sec\frac{kL}{2} - 1\right) \quad (2.52)$$

where

$$k = \sqrt{\frac{P}{EI}}$$

$$\sigma_{max} = \frac{P}{A} + \frac{M_{max}c}{I} \quad (2.53)$$

where

$M_{max} = P(z' + e)$
$c$ = *distance from the central axis about which maximum bending occurs*

Substituting for $M_{max}$ and solving for maximum compressive stress produces the so-called **secant formula** for a simply supported column,

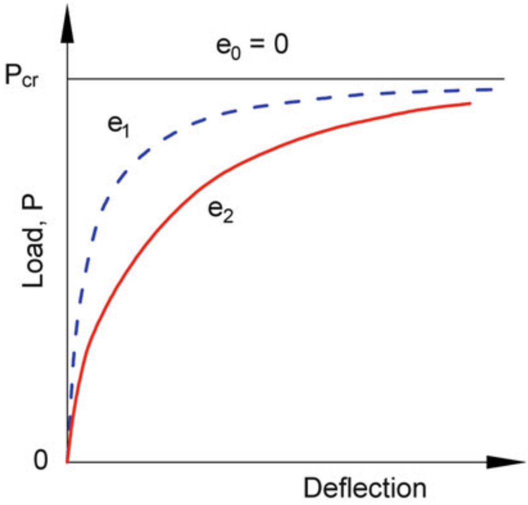


**Fig. 2.49** Load-deflection behaviour for an ideal Euler column ($e_0 = 0$) and two eccentrically loaded columns ($e_1$ and $e_2$)

given by Eq. 2.54. While this formulation finds extensive application in structural engineering, it is important to remember if applying it and similar formulations to rock (assuming that the formulations are indeed valid for rock) that, because the tensile flexural strength of rock is typically 10–30 times less than its compressive flexural strength, tensile flexural strength may be the critical limiting stress rather than the compressive yield stress.

$$\sigma_{max\ comp} = \frac{P}{A}\left[1 + \frac{ec}{r^2}\,sec\left(\frac{L_e}{2r}\sqrt{\frac{P}{EA}}\right)\right] \quad (2.54)$$

where

$\sigma_{max\ comp}$ = *maximum compressive fibre stress at failure (usually taken as yield point for steel and yield strength for light alloys (Young et al. 2012)).*

The solving of Eq. 2.54 requires an iterative approach because of the manner in which the unit load ratio, P/A, appears within the equations. This is aided by computer software.

### 2.8.6 Beam-Columns Subjected to Simultaneous Axial and Transverse Loading

For any condition of column loading, the maximum normal stresses in the extreme fibres of a symmetric column are given by Eq. 2.55:

$$\sigma_{max} = \frac{P}{A} \pm \frac{Mc}{I} \tag{2.55}$$

where

*$M$ = maximum bending moment due to the combined effect of axial and transverse loads*

The difference between the bending moment for a transversely loaded beam and that for an axially loaded column is shown in Fig. 2.50. In the case of a transversely loaded beam, the bending moment is a function of distance, x, along the beam, being at right angles to the direction of deflection. On the other hand, the bending moment for a column is a function of the deflection, δ, and, therefore of distance in the z direction.

The impact of the two sets of bending moments cannot be quantified by simple superposition as the change in deflection due to the axial load disproportionally changes the bending moments. Maximum deflection and bending moments can be calculated from Eqs. 2.56, 2.57, and 2.58, where the terms S(u), F(u) and D(u) are the multipliers to the deflection and moments produced solely by transverse loading.

$$\delta_{max} = \frac{qL^4}{32Et^2} S(u) \tag{2.56}$$

$$M_{\max(abut)} = \frac{qL^2}{12} F(u) \tag{2.57}$$

$$M_{centre} = \frac{qL^2}{24} D(u) \tag{2.58}$$

where

$$S(u) = 5\eta(u)\, X(u)\, \lambda(u)/\tan(u)$$
$$F(u) = X(u)\, u/tan\,(u)$$

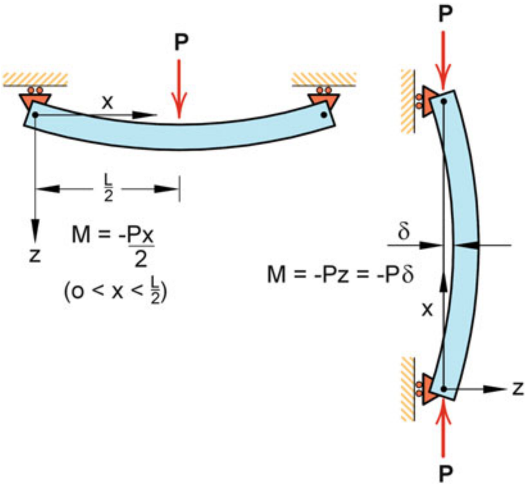


**Fig. 2.50** Comparison between bending moment associated with column loading and bending moment associated with transverse loading

$$D(u) = \Theta(u)\, u/sin\,(u)$$
$$\eta(u) = 12(2\sec u - 2 - u^2)/5u^2$$
$$\lambda(u) = 2(1-cos\ u)\,/(u^2\cos\ u)$$
$$X(u) = 3(\tan u - u)/u^3$$
$$\Theta(u) = 6(u - \sin u)/u^3$$
$$u = L/\left[2(P/EI)^{0.5}\right]$$

### 2.8.7 Thin Plate Subjected to Axial and Transverse Load

When a structure is very long in comparison to its width, the error in analysing its behaviour in the transverse direction as a beam of unit width, rather than as a plate, is minor for points that are at a distance back from the end of the structure that is greater than twice the width of the structure. This type of situation is referred to as **plane strain analysis** because there is assumed to be no strain in the third dimension. At closer distances, end effects associated with the third abutment at the end of the structure become increasing significant and need to be accounted for using plate theory. Hence, in the case of a mine roadway, for example, the immediate roof and floor strata can be analysed as beams provided that the points of interest are more than two roadway widths away from the coal face or an intersection.

Because of the greater likelihood of serious geometrical irregularities and their greater relative effect, the critical stresses for elastic instability actually developed by plates normally fall well short of the theoretical predictions. The discrepancy is usually greater for pure compression than for combined tension and compression and increases with the thinness of the material (Young et al. 2012). Formulae for a range of boundary conditions and loading situations are to be found in specialist texts on statics.

### 2.8.8 Linear Arch Theory

Classical beam theory leads to the conclusion that once the tensile flexural strength of a beam has been exceeded, failure should self-propagate through the beam as its effective thickness is reduced by transverse cracking. However, there are many examples to be found in underground excavations, surface subsidence behaviour, and natural and man-made features of a transversely fractured beam forming a stable arch and spanning a considerably greater distance than predicted by classical beam theory. This behaviour is attributable to the formation of a **voussoir beam**, or **linear arch**, within the beam.

Given the right geometric configuration, a jointed beam will generate a component of lateral thrust at its abutments as it sags. Resistance to this thrust leads to the formation of an internal pressure arch in the beam as depicted in Fig. 2.51. Within a given aspect ratio range, the beam can sustain much higher loads than predicted by simple beam theory. One of the first applications of voussoir beam theory to superincumbent strata behaviour in mining was by Cook et al. (1974) in an attempt to explain rockbursts and rockfalls in deep gold mines in South Africa. Subsequently, the theory has been the subject of a range of laboratory studies, field experience and progressive development by Sterling (1980), Brady and Brown (1985, 2006), Wold and Pala (1986), Seedsman (1987), Pells and Best (1991), Sofianos and Kapenis (1998), Diederichs and Kaiser (1999b), Nomikos et al. (2002) and others.

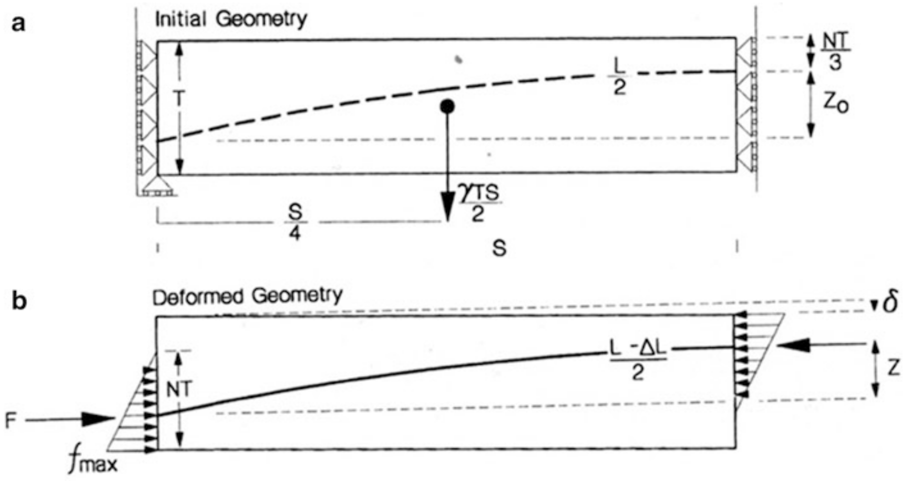
Linear arch theory is premised on assumptions that:

- the beam is horizontal, symmetric, and uniformly loaded;
- the abutment supports are rigid and the joints are very stiff;
- the Poisson's ratio of the rock is zero;
- there is no lateral stress confining the beam prior to its deflection;
- the loading environment results in a state of plane strain.

Brady and Brown (2006) articulated the following linear arch principles associated with jointed or blocky roof beams:

- Roof beds cannot be simulated by continuous elastic beams or plates since their behaviour is dominated by the blocks (voussoirs) generated by natural cross joints or induced transverse fractures.
- Roof bed behaviour is determined by the lateral thrusts generated by the deflection, under gravity loading, of the voussoir beam against the confinement of the abutting rock.
- A voussoir beam behaves elastically (i.e. the lateral thrust-vertical deflection plot is linear and reversible) over the range of its satisfactory performance, the upper limit of which approaches the peak transverse load capacity.
- For a voussoir beam with a low span/thickness ratio, the most likely failure mode is shear failure at the abutments.
- For a beam with high span/thickness ratio, span stability is limited by the possibility of buckling of the beam, with no significant spalling of central or abutment voussoirs.
- A beam with low rock material strength or moderate span/thickness ratio may fail by crushing or spalling of central or abutment voussoirs.

Linear arch problems are statically indeterminate, requiring assumptions regarding the



T = beam thickness
N = some fraction (of beam thickness)
S = length (span) of beam
δ = deflection of beam
$Z_0$ = Initial moment arm (prior to beam deflection)
Z = moment arm for voussoir beam (after deflection)
L = length of linear arch
$f_{max}$ = maximum longitudinal force
F = resultant longitudinal force (thrust)

**Fig. 2.51** Approximations of geometry and forces for a voussoir beam as proposed by Diederichs and Kaiser (1999a)

loading regime. For example, Fig. 2.51b is based on the assumptions that:

- the compression arch developed in the voussoir beam approximates a parabolic shape; and
- triangular load distributions act over the beam abutment surfaces and on the central plane.

Brady and Brown (2006) provide a more in-depth discussion of some of the analytical and numerical techniques used for modelling and analysing voussoir beams. Rock strength, joint friction, joint frequency and compressibility, and the in situ deformation modulus are important input parameters. The selection of input values for these parameters requires particular care as they are potential sources of significant error. Please et al. (2013) provide an analysis of a case in which an originally elastic roof beam transitions into a voussoir beam. The concept is not applicable to situations where joints dip at less than about 75° as the generation of an arch will be prevented by shear failure along these discontinuities.

### 2.8.9 Classical Beam Theory Applications in Ground Engineering

Classical beam theory is very useful for conceptualising the behaviour of rock beams and plates and gaining insight into parameters most likely to determine their response to mining. This can be particularly valuable to a mine operator, for example, for understanding and dealing with specialist advice and for making on-the-job decisions as to how to respond to unexpected or deteriorating roof conditions. However, the opportunity to directly apply classical beam theory to design in underground mining is constrained because the range of ideal conditions on which the theory is based rarely exist in this environment. Natural defects, variations in geometry, eccentric loads, and principal stresses that are not aligned with the axes of the rock structure under analysis are some of the more serious deviations from ideal conditions in the field.

Many of the formulations are of an empirical nature and have been derived with the aid of laboratory and field testing to arrive at calibration factors and failure criteria specific to particular materials. This level of evaluation has yet to be conducted for rock and is unlikely to be undertaken given the availability of alternative design options. Additional limiting factors when applied to rock include the identification of the appropriate end constraint conditions and the quantification of yield strength. Inappropriate end conditions can be a source of significant error in analysis outcomes. Classical beam theory has tended to focus on the performance of structures under compression, whereas rock is much weaker in tension than compression.

Hence, there can be serious limitations associated with using classical beam theory as a design tool in underground mining and it has been superseded in many respects by numerical modelling, especially when the problem is truly three dimensional. Nevertheless, a knowledge of classical beam theory is essential for understanding the underpinning mechanics of strata behaviour and is invaluable for conceptualising behaviour modes, for identifying the parameters that have a dominant influence on these behaviours, and for making informed decisions on-the-job.

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# Excavation Mechanics

# 3

**Abstract**

A mine structure is comprised of three basic building blocks, of which the starting block is an excavation. The development of two separate adjacent excavations results in the formation of the second kind of building block, namely a pillar. The formation of many excavations and pillars requires consideration of a third type of building block, being the surrounding strata. This chapter presents the basic principles of how the rock mass responds to the formation of single and multiple excavations in the same mining horizon.

The changes that take place in the rock mass in the immediate vicinity of an excavation are conceptualised using a number of simple two-dimensional models. The width of an excavation is progressively increased in order to induce caving of the immediate roof strata that, with further increases in excavation span, ultimately result in subsidence of the surface over and outside of the footprint of the excavation. The basic physical and mechanical principles established in Chap. 2 are applied and further developed to account for how mining span, mining depth and the structural and mechanical properties of the superincumbent strata affect the stability of this strata and the maintenance of ground control in the vicinity of active mining faces.

These principles provide the basis for considering three situations where risk may be elevated when employing high percentage extraction mining methods. The situations relate to mining under strong massive strata; to mining in environments subjected to elevated horizontal stress; and to mining at shallow depth. Theoretical and practical aspects associated with each circumstance are discussed. This provides insight into the type of controls required to effectively manage risk.

**Keywords**

Abutment angle • Abutment stress • Angle of draw • Borehole extensometer • Bulking factor • Caving angle • Caving height • Caving mechanics • Caving zones • Seam closure • Constrained zone • Cyclic loading • Discontinuous subsidence • Dolerite sill • Fracturing zones •

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Galvin formula • Goaf consolidation • Horizontal stress • Horizontal stress mitigation • Incremental subsidence • Inrush • Massive strata • Microseismic monitoring • Mining span • Numerical modelling • Periodic weighting • Plug failure • Pressure arch • Primitive stress • Roof fall • Sacrificial roadway • Seam convergence • Seismicity • Shallow mining hazards • Shear angle • Sinkhole • Stress redistribution • Stress shadows • Stress trajectories • Subsidence factor • Subsidence zones • Surface subsidence • Tributary area load

## 3.1 Introduction

A mine structure is comprised of three basic building blocks, of which the starting block is an excavation. The development of two separate adjacent excavations results in the formation of the second kind of building block, namely a pillar. The formation of many excavations and pillars requires consideration to be given to the third type of building block, being the surrounding strata. This chapter presents the basic principles of how the rock mass responds to the formation of an excavation. Coal pillar systems are discussed in a similar manner in Chap. 4, followed by interaction between workings in the same seam and in adjacent seams in Chap. 5. The behaviour of the superincumbent strata is discussed specifically in Chap. 10 but also in some detail in most chapters. This reflects the interdependence between the three fundamental elements, which does not permit each to be discussed in complete isolation.

## 3.2 Excavation Response

The changes that take place when an excavation is developed in a rock mass can be conceptualised using a number of simple two-dimensional models that assume the rock mass is free of defects, behaves elastically and is outside the zone of influence of other excavations. Figure 3.1a shows how the outside surfaces of an isolated portion of this rock mass are loaded by primitive (or virgin) stresses, with the rock inside the boundary of a proposed excavation providing the required internal support to maintain the whole system in a state of equilibrium. When the supporting rock is removed to form the excavation, the stresses on the outer surfaces of the rock mass remain unchanged, but the perimeter of the newly created excavation becomes stress free as shown in Fig. 3.1b. Hence, the initial equilibrium is destroyed and the system of forces must be rearranged to restore equilibrium.

To create the new stress free boundary, stresses must be induced which are equal in magnitude but

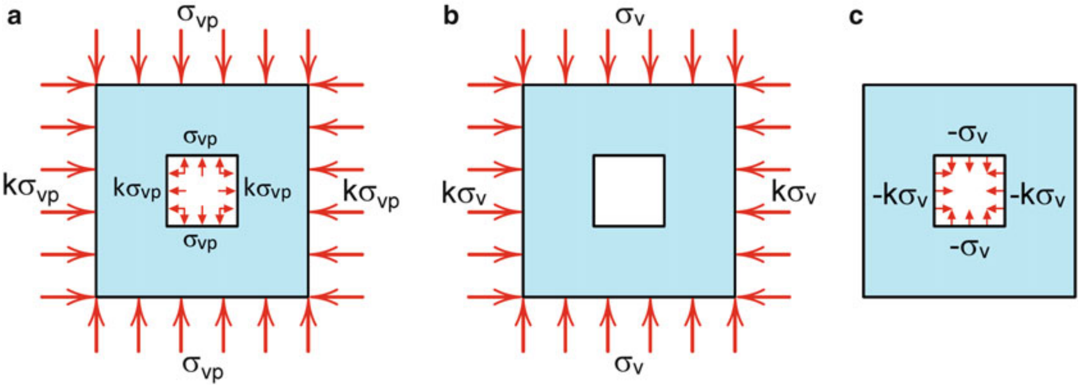


**Fig. 3.1** Primitive (**a**), resultant (**b**) and induced (**c**) stresses around an underground excavation (Adapted from Salamon and Oravecz 1976)

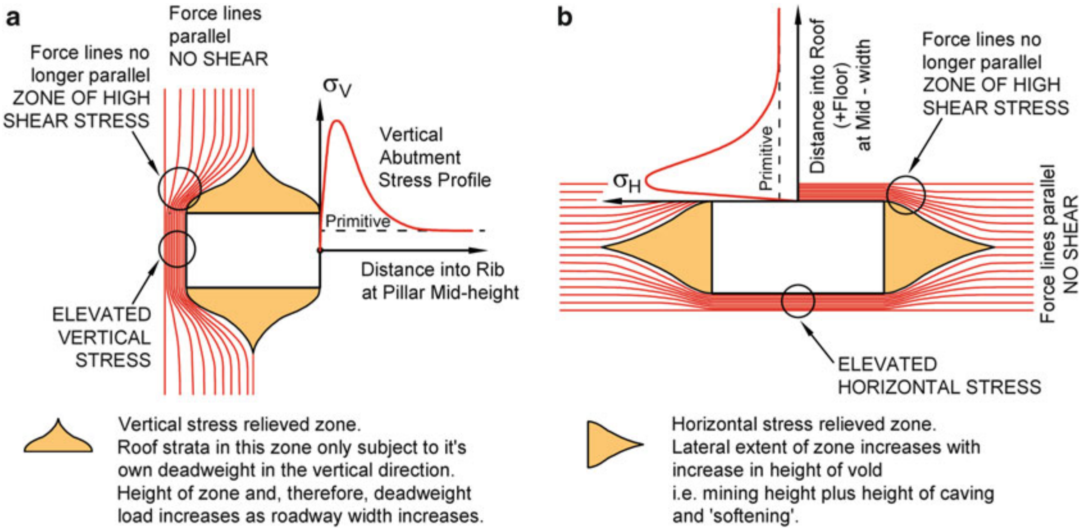

a
Force lines parallel NO SHEAR
Force lines no longer parallel ZONE OF HIGH SHEAR STRESS
σV
Vertical Abutment Stress Profile
Primitive
Distance into Rib at Pillar Mid-height
ELEVATED VERTICAL STRESS
Vertical stress relieved zone.
Roof strata in this zone only subject to it's own deadweight in the vertical direction.
Height of zone and, therefore, deadweight load increases as roadway width increases.
b
Distance into Roof (+Floor) at Mid - width
Primitive
σH
Force lines no longer parallel ZONE OF HIGH SHEAR STRESS
Force lines parallel NO SHEAR
ELEVATED HORIZONTAL STRESS
Horizontal stress relieved zone.
Lateral extent of zone increases with increase in height of void
i.e. mining height plus height of caving and 'softening'.

**Fig. 3.2** Stress trajectories around an excavation

opposite in direction to the compressive primitive stresses that were acting on the excavation boundaries prior to mining. As a result of these induced tensile stresses, shown in Fig. 3.1c, the sides, roof and floor of the excavation can be visualised as being pulled inwards. Because of the upward pull on the floor and the downward pull on the roof, a high vertical stress can be expected in the sidewalls of the excavation.

Plotting of stress trajectories (or streamlines) as shown in Fig. 3.2 provides another means for visualising these responses. The figure shows how an excavation acts as an obstruction to stress flow, with the vertical and horizontal stresses that used to pass through the unexcavated rock mass in parallel trajectories having to deviate around the new excavation. In the case of vertical stress (Fig. 3.2a), this results in:

- increased vertical stress in the sidewalls of the excavation;
- the generation of shear stresses which are a maximum in the vicinity of the excavation corners; and
- a wedge or dome of material in the immediate roof and the immediate floor that, other than for self weight, is relieved of vertical stress.

Analogous conclusions can be drawn with regard to horizontal stress (Fig. 3.2b). It can be concluded from combining these stress trajectory models that:

- increasing excavation width increases vertical compressive stresses in the sidewalls of the excavation; and
- increasing excavation height increases horizontal compressive stresses in the roof and floor strata of the excavation.

In summary, the induced vertical stresses in the immediate roof of an excavation are independent of the size and shape of the excavation and are only a function of depth of mining. The induced horizontal stresses are a function of the shape and size of the excavation, the depth of mining, and the nature and behaviour of the immediate roof, the upper roof and the coal seam. Similar considerations apply in regard to the floor horizon.

Displacement of the rock mass surrounding an excavation is determined by the induced stresses, while rock mass failure is determined by the resultant stresses. Up to the onset of yield in the rock mass, excavation **closure,** or **convergence,** is due solely to elastic rebound. Although horizontal compressive stresses are induced in the roof and floor strata of an excavation, the resultant horizontal stresses may still be tensile, depending on depth, excavation geometry and primitive horizontal to vertical stress ratio.

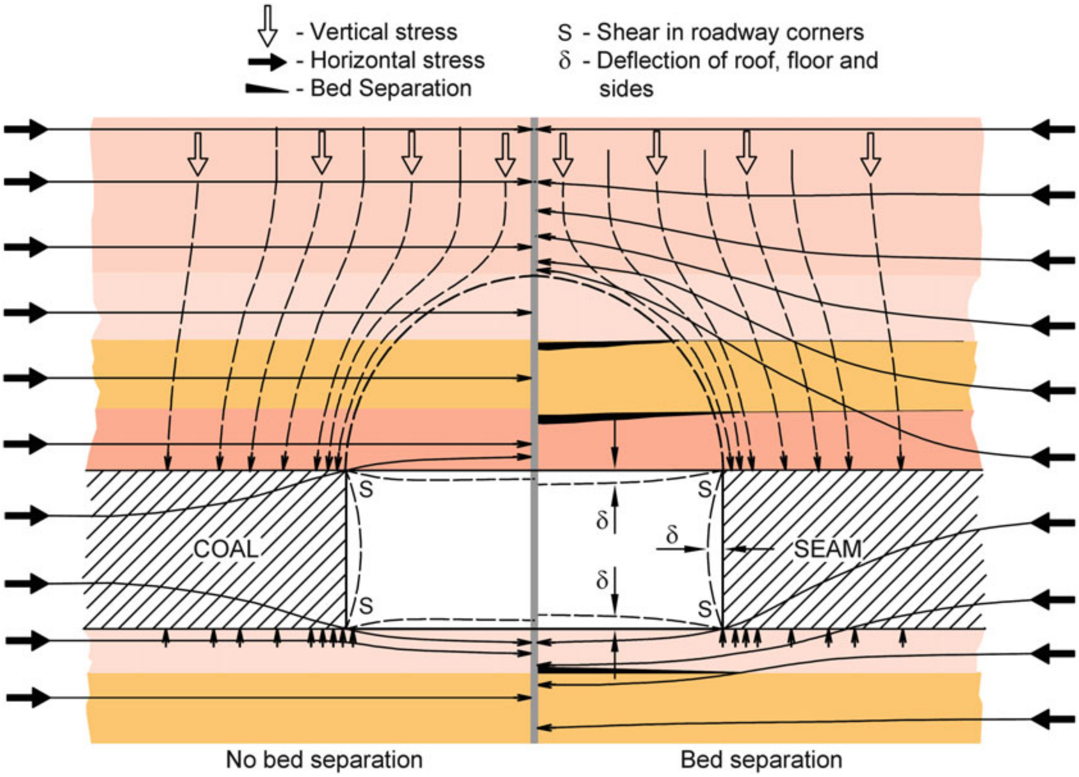


**Fig. 3.3** Redistribution of stress around a narrow excavation, before and after delamination of immediate roof and floor strata (Partially adapted from Menzies c1970)

Figure 3.3 shows in more detail the manner in which stresses and strains are distributed around an excavation prior to the onset of caving of the immediate roof. The vertical stress distribution gives rise to the formation of a dome shaped zone, referred to as a **pressure arch**, with the weight of the strata outside of the pressure arch being transferred to the abutments of the excavation. The strata within the pressure arch effectively constitute a decoupled immediate roof for the excavation, being vertically destressed and loaded transversely by only its own weight and axially by lateral forces.

In coal mines, the immediate roof and floor strata are usually bedded due to the sedimentary origin of coal deposits. Bedding planes are characterised by low to zero tensile strength normal to the bedding planes and low shear strength relative to that of intact rock. Hence, bedding planes constitute potential slippage planes and can effectively divide the roof strata into an assembly of thin rock beams, thereby permitting the immediate roof to sag under its own weight. The sense of slip causes inward displacement towards the centreline of the span and decreases with height into the roof, so that there is a tendency for the beds to delaminate, or decouple, in both the immediate roof and the immediate floor. In the case of the immediate roof strata, bed separation results in a loss of load sharing with upper beds and the transfer of horizontal stress to higher horizons in the roof as shown on the right hand half of Fig. 3.3. Stress transfer to deeper horizons in the floor is not so prevalent but does occur with the onset of floor heave.

Bedded strata can be conceptualised as a series of plates. If an excavation is long compared to its width, roof strata remote from the ends of the excavation can be visualised as behaving as beams when subjected to transverse load and as columns when subjected to axial load. Hence, classical beam theory and linear arch theory find application for analysing the behaviour of the immediate roof of an excavation, albeit that limitations are associated with these approaches (see Sect. 2.7 and Chap. 7).

It is important to appreciate that ground failure is not induced by just an increase or decrease in one component of stress but rather by a change in deviator stress (see Sect. 2.6.2). The biggest change in deviator stress results from the total removal of confinement, which causes a triaxial state of stress to revert to a uniaxial state of stress. The formation of an excavation results in the rock mass surrounding the excavation undergoing this transition. Hence, whilst failure is often attributed to high abutment stress, reduction in confinement is more likely to be the primary cause.

## 3.3 Caving Mechanics

### 3.3.1 Basic Principles

It is convenient to introduce the basic principles of caving mechanics and goaf reconsolidation by considering the simple case of a single (isolated) total extraction panel of width, W, assumed to be at low to moderate depth, H, say less than 200 m. Although the principles under consideration are relevant to mining at greater depth, they can rarely be extrapolated directly as a design tool to deeper situations because they fail to adequately reflect the mechanics of the loading system, in particular, the stiffness of the superincumbent strata.

The minimum plan dimension, or lateral dimension, of an excavation is the critical dimension that controls the response of the rock mass. In underground coal mining, deliberate caving of an excavation is usually undertaken only as mining retreats from a panel. This means that the length of the excavation is the critical dimension up until the point where the excavation takes on a square shape. Thereafter, the excavation width becomes the critical dimension. The minimum excavation dimension at any point in time is referred to as the **span**.

Ultimately, with increasing excavation span, the immediate roof loses its capacity to bridge across the excavation and begins to cave. If failure is associated with shear at the abutments then near vertical failure surfaces may extend for several metres up into the roof. Otherwise, as illustrated in Fig. 3.4, the immediate roof caves out over the excavation. Because caved rock strata consist of blocks of rock which rotate when they fall, the caved rock material occupies a greater volume than when in situ. This behaviour is known as **bulking**. Weak laminated strata, such as shales and mudstones, generally cave at a steeper angle and bulk less than stronger and more massive strata, such as sandstone. With increasing excavation span, the point is reached where due to the combined effects of caving, bulking, lowering of the roof and uplift of the floor, the caved material comes into contact with

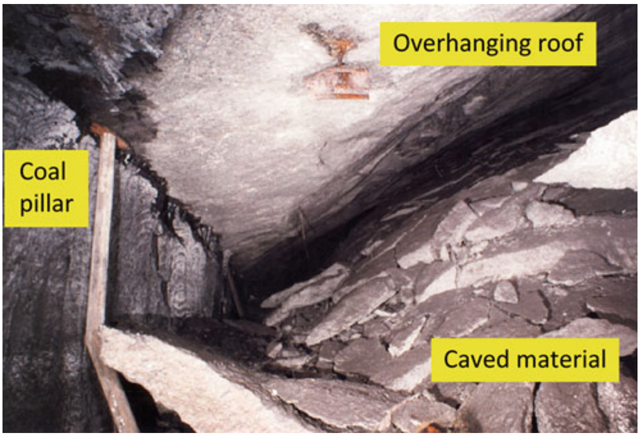


**Fig. 3.4** An example of how the immediate roof strata cantilevers out over the goaf

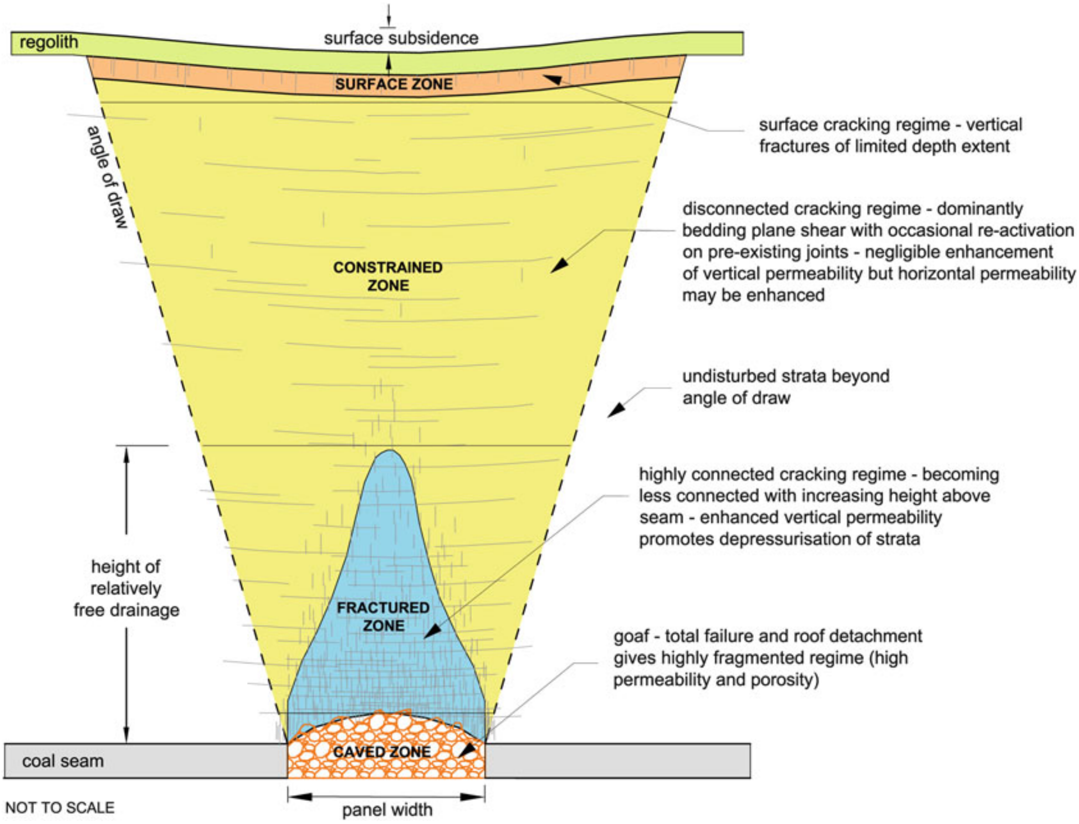

regolith
surface subsidence
SURFACE ZONE
surface cracking regime - vertical fractures of limited depth extent
angle of draw
CONSTRAINED ZONE
disconnected cracking regime - dominantly bedding plane shear with occasional re-activation on pre-existing joints - negligible enhancement of vertical permeability but horizontal permeability may be enhanced
undisturbed strata beyond angle of draw
height of relatively free drainage
highly connected cracking regime - becoming less connected with increasing height above seam - enhanced vertical permeability promotes depressurisation of strata
FRACTURED ZONE
goaf - total failure and roof detachment gives highly fragmented regime (high permeability and porosity)
coal seam
CAVED ZONE
NOT TO SCALE
panel width

**Fig. 3.5** A conceptual four zone model of caving and fracturing above an excavation (Courtesy of Dr Colin Mackie)

the strata above and provides support to it. This defines the **caving height** and the limit of the **caved zone**, shown in Fig. 3.5.

The overlying strata continue to sag as excavation span increases but the support and cushioning provided to this strata by the caved material prevents them from unravelling and increasing appreciably in volume. Nevertheless, considerable fracturing and bedding plane shear stills occurs within the sagging strata, resulting in a well developed and connected vertical and horizontal fracture network that becomes less extensive and connected with distance above the excavation. This network defines the **fractured zone** (Fig. 3.5).

The lateral extent of roof strata sag increases with distance above the excavation, resulting in a decreasing rate of deflection, or curvature, in the upper roof strata and a corresponding reduction in shear and bending stress. This zone is referred to as the **dilated zone** in some models.

Given sufficient depth, a point is reached where the tensile stresses become too low in the upper strata to cause joints to open or new vertical fractures to develop on a regular or continuous basis. Horizontal fracture planes are still likely to be activated due to sagging strata sliding past each other but the magnitude of these displacements also reduces as curvature decreases. The zone in which this behaviour occurs is referred to as the **constrained zone** (Fig. 3.5), and is characterised by strata that have not suffered a significant alteration of their physical properties. Stress measurements, field observations of subsidence behaviour and numerical modelling show that the constrained zone can sustain elevated levels of horizontal stress.

Strata response close to the surface is influenced by the surface being in an unconfined state. In flat topography, this response essentially mirrors that of a beam (or plate). Within this

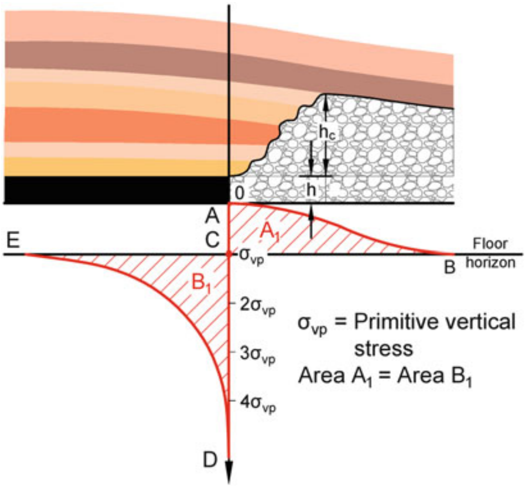


**Fig. 3.6** Stress distribution at floor horizon around a caved excavation

so-called **surface zone** (Fig. 3.5), ground behaviour is characterised by bedding plane shear and tensile and compressive cracking of limited vertical extent.

Figures 3.5 and 3.6 show these various zones to be bounded by a conceptual **angle of draw.** The angle of draw defines the limit of the vertical component of surface displacement and, hence, the extent of the subsidence trough. It is the angle between two lines drawn from the edge of the mine workings, one a vertical line and the other a line to the limit of **vertical displacement, $V_z$,** on the surface. Although the concept of a line demarcating the boundary between moving and stationary ground is theoretically untenable, nevertheless the approach finds extensive application and is helpful in conceptualising ground behaviour. Angle of draw is discussed further in Chap. 10.

There is no one universally accepted model of subsurface behaviour. While all models are based on similar principles, there is considerable variation in terminology, the number of zones, and the proposed thickness of each zone. This is usually a reflection of the site specific lithology and the nature of the subsidence impact being assessed. Subsurface subsidence behaviour models are discussed further in Sect. 10.3.

Once the caved material comes into contact with the overlying strata, further increases in excavation span cause progressive compaction of the caved material, thereby increasing its stiffness and the level of support it provides to the superincumbent strata. If the excavation span is sufficiently wide and the depth of mining not too great, the stiffness of the superincumbent strata will be reduced to zero, resulting in the full weight of the overburden being carried by the caved material and transferred through to the floor at some distance back into the goaf, as shown in Fig. 3.6. Vertical displacement of the surface reaches a maximum value above this point.

The panel span-to-depth ratio, W/H, at which the stiffness of the overburden is reduced to zero corresponds with the vertical surface displacement above an isolated panel reaching its maximum possible value and is referred to as the **critical** width-to-depth ratio, $W_c/H$. Larger panel width-to-depth ratios are referred to as being **supercritical** and smaller panel width-to-depth ratios as being **subcritical**. It is very common in subsidence engineering to normalise vertical displacement, $V_z$, by dividing it by mining height, h, and then plotting this ratio ($V_z/h$) against W/H. This is an example of the care that must be taken when working with normalised, or dimensionless, relationships since mining height has minimal influence on subsidence at subcritical panel width-to-depth ratios.

That portion of the weight of undermined strata not carried by the floor of an excavation at a given W/H ratio is transferred onto the abutments of the excavation, giving rise to abutment stress. Hence, in the two-dimensional representation shown in Fig. 3.6, the area of the destressed zone ‘ABC’ has to equal the area of the elevated stress zone ‘CDE’. The actual shape and size of these zones depends on many factors, including the caving and compaction characteristics of the immediate roof, the stiffness of the overburden, the excavation width-to-depth ratio and the extraction height. If the rock mass is considered to be elastic and homogeneous then, in theory, the tangential stress at the right angled corner of a rectangular opening approaches infinity. This can be managed by profiling the corners of the excavation. In practice, the corners of rectangular excavations yield, causing the peak abutment stress front to migrate into the solid. Guttering can be an expression of this yielding process.

Weak strata, such as shales and mudstones, cave at a steeper angle than stronger strata such as sandstones and conglomerates. Typically, the overall caving angle, β, (measured from the mining horizon) is observed from the goaf edge to be in the range of 65°–80° for weak strata and 55°–65° for stronger strata. Some massive strata may cantilever a considerable distance out into the goaf before breaking and relieving abutment stress. This cyclic process of caving, convergence and recompaction is commonly referred to as **periodic weighting**.

Bulking and compaction characteristics determine the rate that the goaf generates support to the undermined strata, and the distance back into the goaf to restoration of full overburden support. Caving and subsidence progress relatively uniformly through bedded and weak strata, while more massive and strong strata often subside in a discontinuous manner as a series of discrete blocks that separate at distinct horizons within the superincumbent strata (see for example, Hardman 1971; Galvin et al. 1982; Mills and O'Grady 1998). The **bulking factor, k**, is defined as the ratio of the total volume of the caved material to its solid volume. For situations where caving progresses uniformly up into the roof strata, **the initial bulking factor, $k_i$**, is given by Eq. 3.1.

$$k_i = \frac{h + h_c}{h_c} \tag{3.1}$$

*where*

*h = mining height*
*$h_c$ = maximum initial height of caving above mining roof*

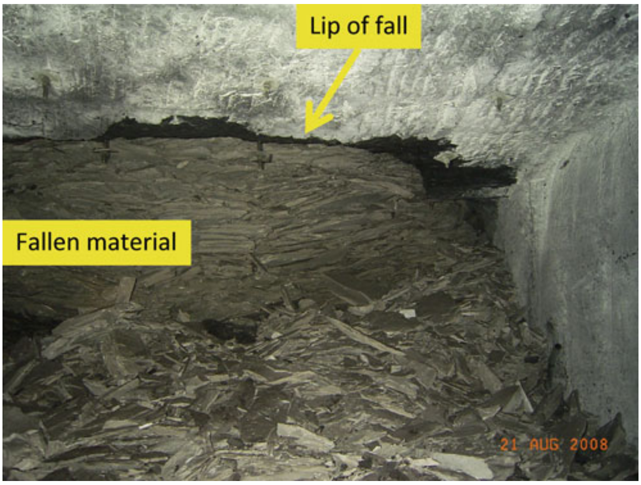
hence

$$h_c = \frac{h}{k_i - 1} \tag{3.2}$$

Equation 3.2 shows that mining height has a significant influence on the height of caving. A steep caving angle and a low initial bulking factor, $k_i$, of the order of 1.1–1.3, are usually associated with highly laminated strata because they tend to fall like a deck of cards, with minimal rotation and relative displacement of blocks, as illustrated in Fig. 3.7. Caving angles are flatter and block rotation occurs to a much greater extent in stronger and more massive strata, as illustrated in Fig. 3.8, with initial bulking factors for sandstone, for example, being in the range of 1.4–1.5 (Galvin et al. 1982). Although in reality caving may be interrupted by strong or massive beds, the concept of bulking factor still provides a foundation for understanding goaf behaviour and reconsolidation and the loading of support systems in total extraction mining.

The load-deformation relationship for goaf material is not linear, with compaction causing a reduction in bulking factor and an increase in stiffness. The hostile nature of a goaf environment presents major challenges to measuring consolidation and strain hardening

**Fig. 3.7** A fall of ground at an intersection, illustrating attributes of a steep caving angle and a low initial bulking factor associated with weak laminated strata



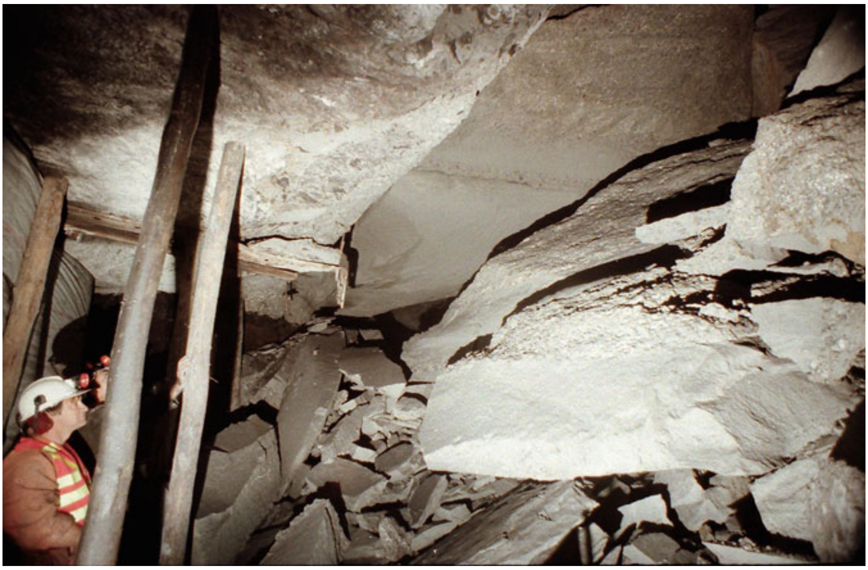


**Fig. 3.8** Caving of a moderately strong massive roof, illustrating attributes of a flat caving angle and a high initial bulking factor

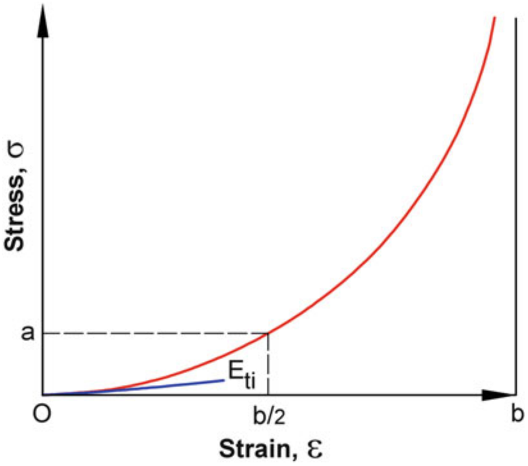


**Fig. 3.9** Stress-strain characterisation of goaf backfill (After Ryder and Wagner 1978)

characteristics in the field, with attempts by Jacobi (1956), Wade and Conroy (1980), Haramy and Fejes (1992), Campoli et al. (1990), Wang and Peng (1996) and Kelly et al. (1998) and others producing variable outcomes.

In the absence of experimental work on the physical properties of goaf material, Salamon (1966) explained goaf consolidation and surface subsidence behaviour by adopting a soil mechanics relationship between porosity and pressure given by Kezdi (1952). The behaviour of backfill in goaf areas also provides insight into goaf consolidation behaviour. Ryder and Wagner (1978) described the consolidation behaviour of goaf backfill by the stress-strain relationship shown in Fig. 3.9 and concluded that Eqs. 3.3 and 3.4 describe this relationship surprisingly well.

$$\sigma_f = \frac{a\varepsilon}{b - \varepsilon} \tag{3.3}$$

where

$\sigma_f$ = *fill reaction stress*
$b$ = *maximum compaction of backfill material*
$a$ = *fill reaction stress at a strain value of* $0.5b$

Equation 3.3 can be reduced to:

$$\sigma_f = \frac{E_{ti}\varepsilon}{\left(1 - \frac{\varepsilon}{\varepsilon_m}\right)} \tag{3.4}$$

where

$\sigma_f$ = *fill reaction stress*
$E_{ti}$ = *initial tangent modulus for the stress–strain curve of the fill material*
$\varepsilon$ = *compaction strain at any point in time*
$\varepsilon_m$ = *the maximum possible strain that can be developed in the fill*

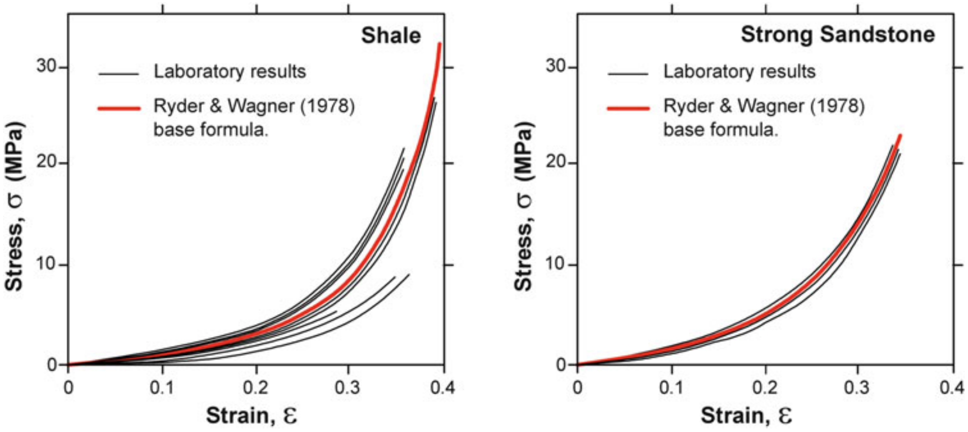


**Fig. 3.10** Comparison between analytically and laboratory derived stress-strain relationships for two types of goaf material (Adapted from Pappas and Mark 1993)

Salamon (1990) showed that there was a reasonable body of evidence, including that of Ryder and Wagner (1978), to suggest that Eq. 3.5 borrowed from studies of granular material may be a credible descriptor of the relationship between bulking factor and overburden pressure:

$$k = \frac{p + k_i p_c}{p + p_c} \tag{3.5}$$

where

$k_i$ = *initial bulking factor*
$k$ = *bulking factor at any point in time*
$p$ = *applied overburden stress*
$p_c$ = *a material constant ranging from 0.5 MPa to 5.0 MPa for coal measure rocks*

Rearranging Eq. 3.5 produces the expression given by Eq. 3.6 for applied overburden stress:

$$p = \frac{p_c (k_i - k)}{k - 1} \tag{3.6}$$

Salamon noted that Eq. 3.4 could be equated with Eq. 3.6, thus enabling the material constant, $p_c$, to be defined by Eq. 3.7.

$$p_c = E_{ti} \varepsilon_m \tag{3.7}$$

Pappas and Mark (1993) reported that numerical modellers have used estimates of goaf modulus that range from around 7 MPa (~1000 psi) to over 2.1 GPa (~300,000 psi), with such wide variations greatly affecting the outcomes of the numerical analyses. Based on laboratory scaled tests, the authors developed the formulae given by Eqs. 3.8 and 3.9 for secant and tangent modulus of goaf material (note that these formulae cannot be converted to metric equivalents by simply expressing stress in metric units). They concluded that although these formulae could not be assumed to describe the moduli of all goaf material types, they produced outcomes that corresponded reasonably well with theoretical solution curves developed by Ryder and Wagner (1978) and Salamon (1990). Two examples of this correlation are shown in Fig. 3.10.

$$E_s = 2.36\sigma + 1360 \tag{3.8}$$

$$E_t = 0.00181\sigma^2 + 9.33\sigma + 294 \tag{3.9}$$

where

$E_s$ = *Secant modulus (psi)*
$E_t$ = *Tangent modulus (psi)*
$\sigma$ = *Stress level due to overburden (psi)*

The magnitude and distribution of floor pressure beneath the goaf is also a function of the stiffness of the upper roof strata, since it is the displacement of this strata that provides the driving force for compaction. The displacement is determined by the elastic modulus, span and

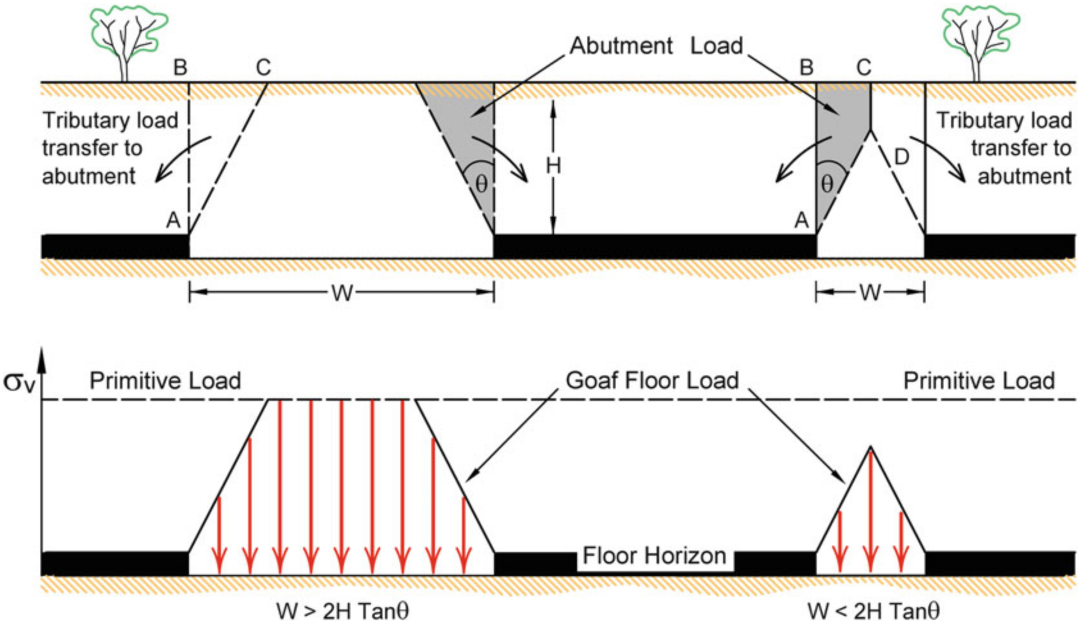


**Fig. 3.11** Simplified model of load transfer around an isolated excavation (Adapted from Salamon 1991a, after King and Whittaker 1971)

thickness of the individual overlying stratum and how they behave as a composite. Quantifying the manner in which the relative stiffnesses of the goaf material, the overlying undermined strata and the surrounding strata determine the load distribution around an excavation is complex and requires the use of numerical modelling.

Nevertheless, a simplified two-dimensional model proposed by King and Whittaker (1971) provides a basis for conceptualising how abutment load is generated around an isolated panel, although as suspected by Mark and Bieniawski (1987), this model is not as straightforward as implied by its developers. The model assumes the superincumbent strata shears off over the goaf at some **shear angle**, θ, measured from the vertical, as shown in Fig. 3.11. It forms the basis of a number of others in which the term 'shear angle' is referred to variously as the **abutment angle**, ϕ, measured from the vertical, and the **angle of caving**, β, and the **angle of break**, β, where β is measured from the horizontal. The abutment angle concept equates abutment load to the equivalent weight of a wedge of rock projecting off the abutment at an angle, ϕ. The weight of this rock is apportioned to the sides of an excavation in accordance with tributary area theory, which distributes overburden load to a pillar or abutment on the basis of its area of influence as defined by the loci of mid-points to adjacent pillars and abutments (see Fig. 3.11 and also Sect. 4.3.2).

Figures 3.12 and 3.13 show examples of the concept of shear angle in practice above longwall panels at shallow depth in weak immediate roof strata. In the case of Fig. 3.13, overall shear angle ranged from 18.5° to 23°.

According to these models, once the shear angle daylights at the surface, any further increase in span results in full overburden load being transmitted through the goaf to the floor of the excavation. Therefore, the daylight span should correspond to the critical span at which full surface subsidence, or vertical surface displacement, develops (ignoring additional time-dependent compaction and settlement, which typically makes up about 10 % of the final vertical displacement).

There are a number of limitations associated with applying the concept of shear angle to

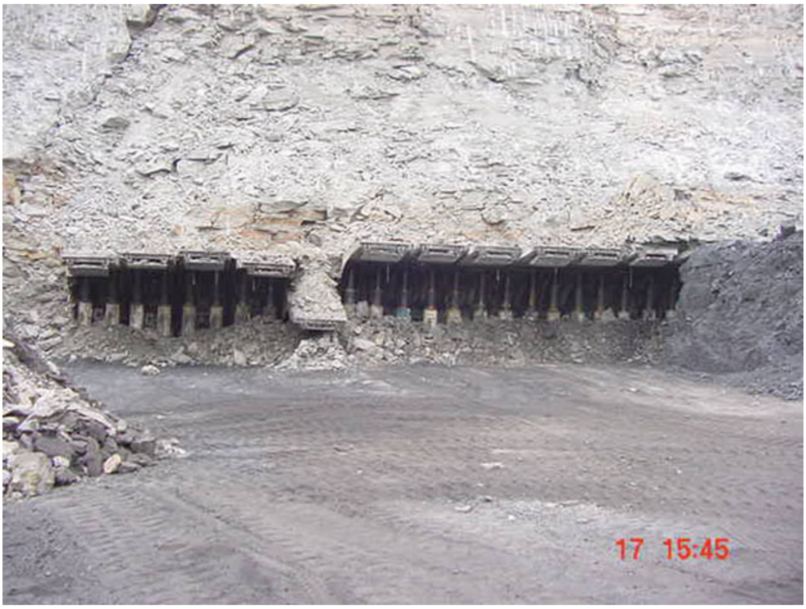


**Fig. 3.12** Exposed goaf showing angle of break at the side abutment of a shallow, supercritical width, longwall panel in a weak immediate roof strata environment at a depth of approximately 70 m

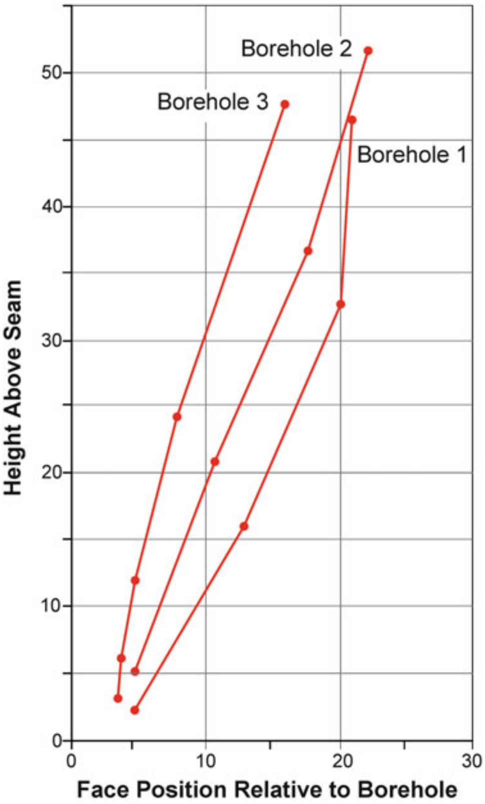


**Fig. 3.13** Caving behaviour in weak shale roof above a longwall panel at a depth of mining of 115 m as determined from borehole extensometers (After Galvin et al. 1982)

calculate abutment load and these can affect the reliability of design outcomes. The limitations include that the model is insensitive to differences in the caving behaviour of individual strata (which can be extreme); it has no regard to **discontinuous subsidence**, whereby subsidence is impeded by competent strata forming a bridge somewhere within the superincumbent strata; it may not adequately approximate the shape of a pressure arch; and it has no regard to the stiffness of the superincumbent strata. The potential for these limitations to arise is reflected in the range in shear angles for individual strata of 16°–29° measured (from the vertical) in South Africa using borehole extensometers (Galvin et al. 1982), and 5°–35° reported by Peng (2008) on the basis of reviewing literature. Internationally, a range of surface subsidence data, such as that shown in Fig. 3.14, indicate that a panel width-to-depth ratio of at least 1 is required to induce full subsidence over an isolated coal mine excavation. This corresponds to an overall shear angle of 26.5°.

The most significant limitation associated with equating abutment load to an overall shear angle, or so-called abutment angle, is that the concept has no regard to the stiffness of the

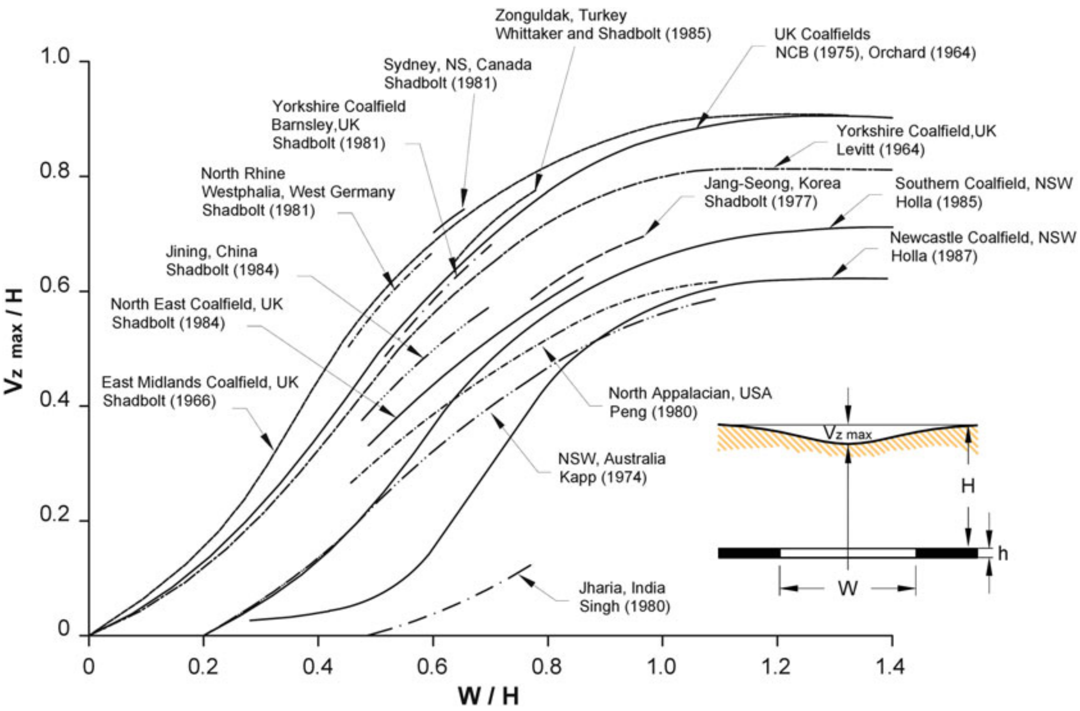


**Fig. 3.14** Influence of extraction panel width-to-depth ratio on maximum vertical surface displacement, $V_{z\ max}$, expressed as a fraction of mining height, for isolated total extraction panels (Adapted from Whittaker and Reddish 1989, page 355, copyright Elsevier 1989)

superincumbent strata, although there have been attempts to take this into account by using different angles for different mining districts. Profiles of vertical displacement at the surface (often generically referred to as 'surface subsidence') are a reflection of the stiffness of the superincumbent strata and, therefore, give valuable insight into the distribution of superincumbent strata load. This is illustrated by the vertical surface displacement profiles shown in Figs. 3.15 and 3.16, which are associated with two sets of 210 m wide longwall panels under not very dissimilar geological conditions, one at a depth of ~80 m and the other at a depth of ~500 m.

When the depth of cover is low (typically less than 150 m) and the total excavation width-to-depth ratio, W/H, for an individual panel is high (typically, at least one and often higher), the stiffness of the superincumbent strata over a shallow excavation can reduce to zero as it is being extracted, resulting in vertical surface displacement over that panel developing virtually independently of that over adjacent panels. The abutment load on the interpanel pillars is relatively low because the depth of cover is shallow and because the superincumbent strata over the flanking excavations does not dome and form a bridge. This results in near symmetrical profiles of vertical surface displacement, such as those shown in Fig. 3.15, as soon as each panel is extracted. In these circumstances, compression of the interpanel pillars (chain pillars) and their immediate roof and floor strata makes only a minor contribution to vertical displacement and over 90 % of the final vertical displacement at a surface point is usually reached within weeks of it being undermined. The measured vertical surface displacement above interpanel pillars in these circumstances may largely reflect interaction of neighbouring subsidence troughs, illustrated in Fig. 3.17, rather than compression of the pillar system and surrounding strata.

The situation is quite different at depth. Figure 3.16 shows that limited vertical surface

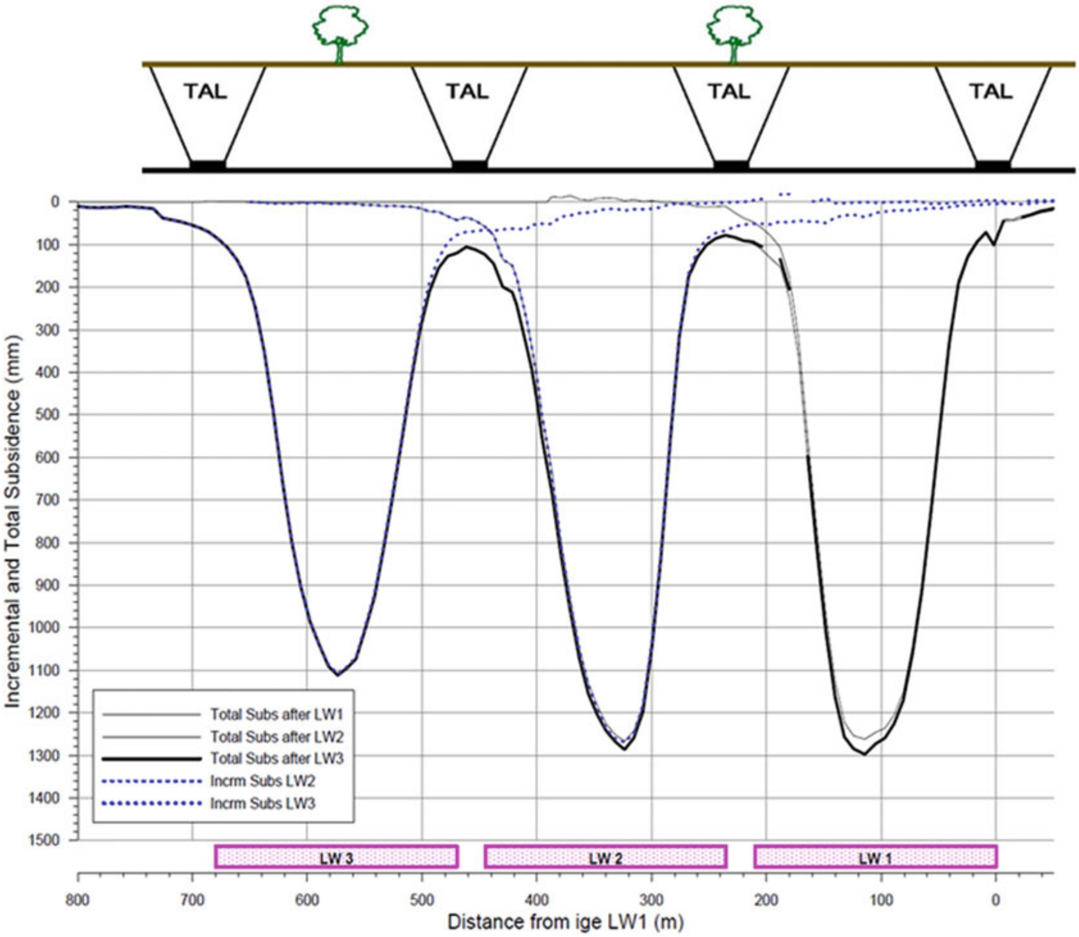


**Fig. 3.15** Vertical surface displacement profiles over 210 m wide longwall panels at a depth of around 80 m (W/H = 2.6) showing how maximum surface displacement develops virtually independently of subsequent panel extraction at shallow depth, consistent with tributary area load (TAL) based on the concept of an abutment angle

displacement occurred over the first longwall panel, being LW 401, when it was extracted. Extraction of LW 402 resulted in a large step increase in vertical displacement over LW 401. This additional displacement is referred to loosely as **incremental subsidence**. The overall vertical surface displacement profile is found by summing the incremental subsidence profiles. The pattern of change in the incremental subsidence profiles as more longwall panels are extracted is evidence of a progressive reduction in the stiffness of the superincumbent strata, resulting in increased compression of the pillar system and surrounding roof and floor strata. Vertical surface displacement over LW 401 continued to increase in increments during extraction of at least the next four longwall panels, albeit at a diminishing rate. Once the stiffness of the overburden had been reduced to zero, incremental vertical displacement reached a steady state.

The behaviours shown in Figs. 3.15 and 3.16 are similar to those associated with Longwall 103 at Gordonstone Colliery and Longwall 28 at Appin Colliery, which were the sites of microseismic research reported by Hatherly et al.(1995), Kelly et al.(1998) and Kelly and Gale (1999). Figure 3.18 shows the profile of

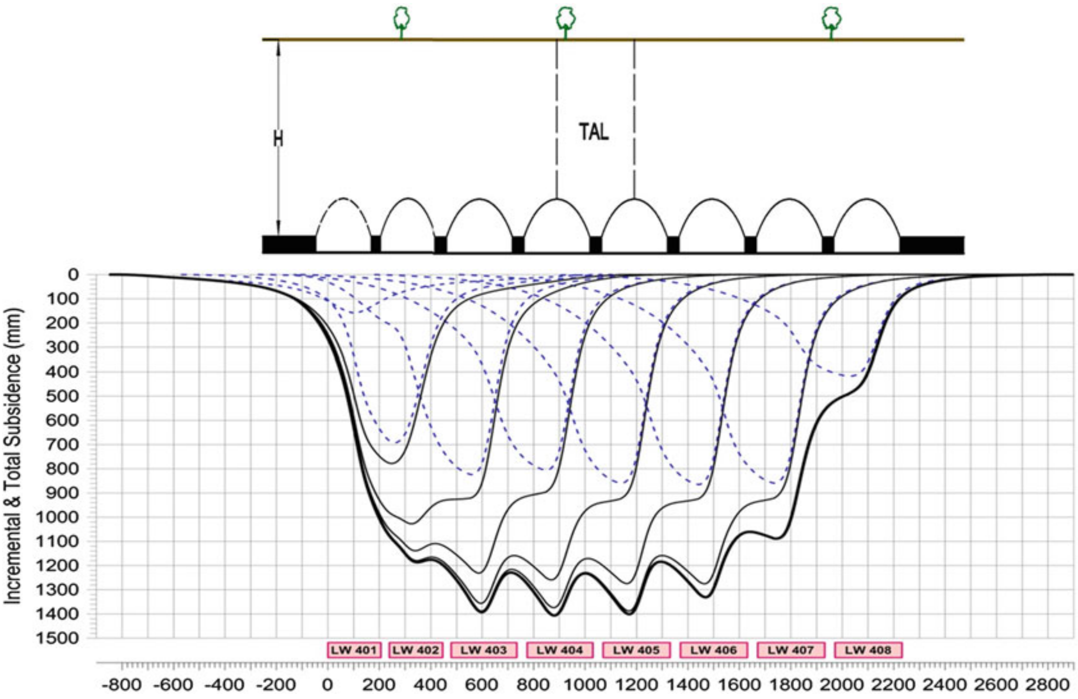


**Fig. 3.16** Vertical surface displacement profiles over 210 m wide longwall panels at a depth of around 500 m (W/H = 0.42) showing how maximum vertical surface displacement develops incrementally at depth as subsequent panels are extracted and not in accordance with tributary area load (TAL) based on the concept of abutment angle

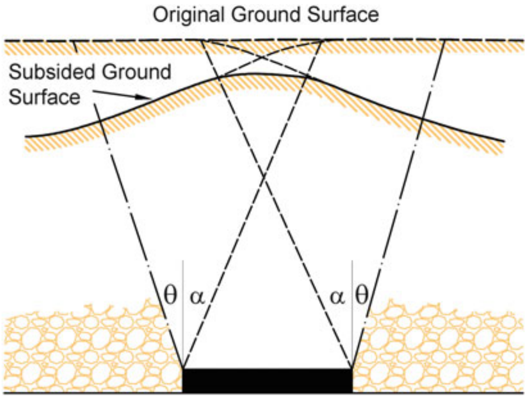


**Fig. 3.17** Illustration of how overlap in subsidence profiles results in vertical displacement over an interpanel pillar

vertical surface displacement and microseismic activity plots associated with extracting a 3 m thick seam at a depth of around 235 m utilising a 250 m wide longwall at Gordonstone Colliery. Kelly et al. (1996) and Hatherly and Luo (1999) report that Longwall 103 was a benign setting where the geology above and below the seam had relatively uniform properties, there were no geological structures, and the horizontal stress field was not severely distorted by previous mining panels. Thus, the researchers considered that they were able to observe what might be called 'classical' behaviour. The immediate roof and floor were particularly weak, with UCS values of only 5–15 MPa, while some bands in the upper roof strata had UCS values of about 50 MPa (Kelly et al. 1998). Of particular note is that the upper 70 m of overburden comprised unconsolidated material. This could be anticipated to both function as a surcharge load and to increase the effective panel width-to-depth ratio (due to the reduced thickness of solid rock cover).

The microseismic events were located to an accuracy of about 5 m and, as shown in Fig. 3.18b, d, confined mainly to within 120 m above the mining horizon and 50 m below it. The events were located within an envelope rising above the gateroads at an angle of about 16° to the vertical and sweeping ahead of the face on an arcuate shape (Fig. 3.18c). At seam level in the centre of the panel, the envelope was some 70 m ahead of the face and rose at an angle of about 50° to the vertical over the face (Fig. 3.18d). Pore pressures began to increase markedly at around 200 m ahead of the face, at distances coinciding with the onset of the microseismic activity. Kelly and Gale (1999) suggested that such rises in pore pressure contribute to the rock mass fracturing at such long distances ahead of the longwall face.

The pattern of seismicity was remarkably symmetric, with events located equally on the tailgate and maingate sides of the panel. Microseismic activity did not necessarily coincide with any marked deterioration in ground conditions in mine roadways or on the longwall face and there was no suggestion of a cyclic pattern of failure. The microseismic events were attributed to shear failure, with failure planes parallel to and rising over the longwall face. Bedding plane shear, if it was occurring, did not cause seismic activity sufficient for it to be observed. These microseismic monitoring outcomes, supported by profiles of vertical surface subsidence, are consistent with the residual stiffness of the superincumbent strata being reduced to a negligible value over each individual longwall panel as soon as it was undermined, so that the panels largely acted independently of each other. In circumstances such as these where mining is taking place at relatively shallow depth beneath strata that

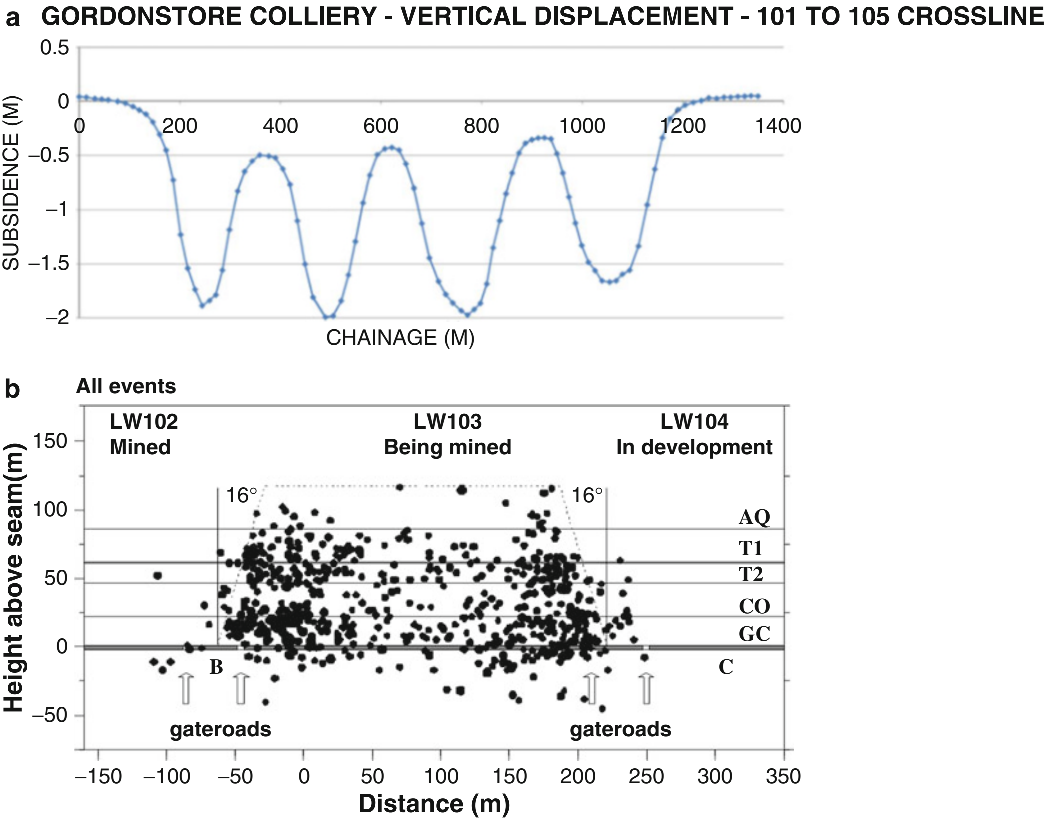


**Fig. 3.18** Surface subsidence profile and microseismic event location plots associated with extracting Longwall 103 at Gordonstone Colliery. (**a**) Transverse vertical surface displacement profile ($H = 235$ m, $H_{\text{solid rock}} = 165$ m, $W/H_{\text{solid rock}} = 1.5$), (**b**) Vertical section across panel, (**c**) Plan view, (**d**) Vertical section along panel ((**a**) Courtesy Gordonstone Colliery; (**b**), (**c**) and (**d**) after Hatherly and Luo 1999)

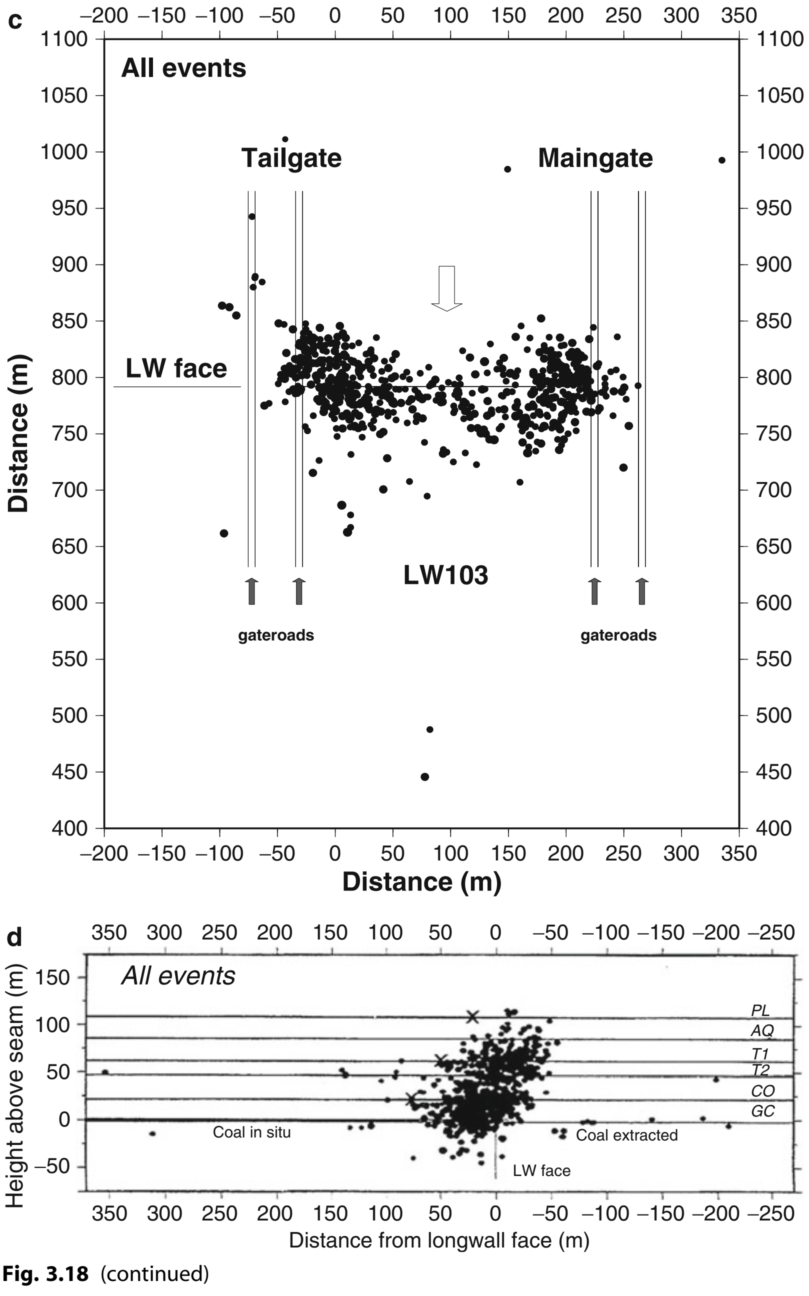


**Fig. 3.18** (continued)

readily caves, there is considerable scope to apply the abutment angle concept.

The ground behaviour at Gordonstone Colliery contrasted with that monitored at the much deeper Appin Colliery. Appin Colliery extracted a 2.3 m thick seam at a depth of about 500 m utilising a 200 m wide longwall face, resulting in individual panel width-to-depth ratios, W/H, of only 0.4–0.5. The superincumbent strata comprised an alternation of sandstone units up to 120 m thick with interspersed claystone and shale units up to 25 m thick. This strata was described in general terms as constituting medium to strong roof conditions (Kelly

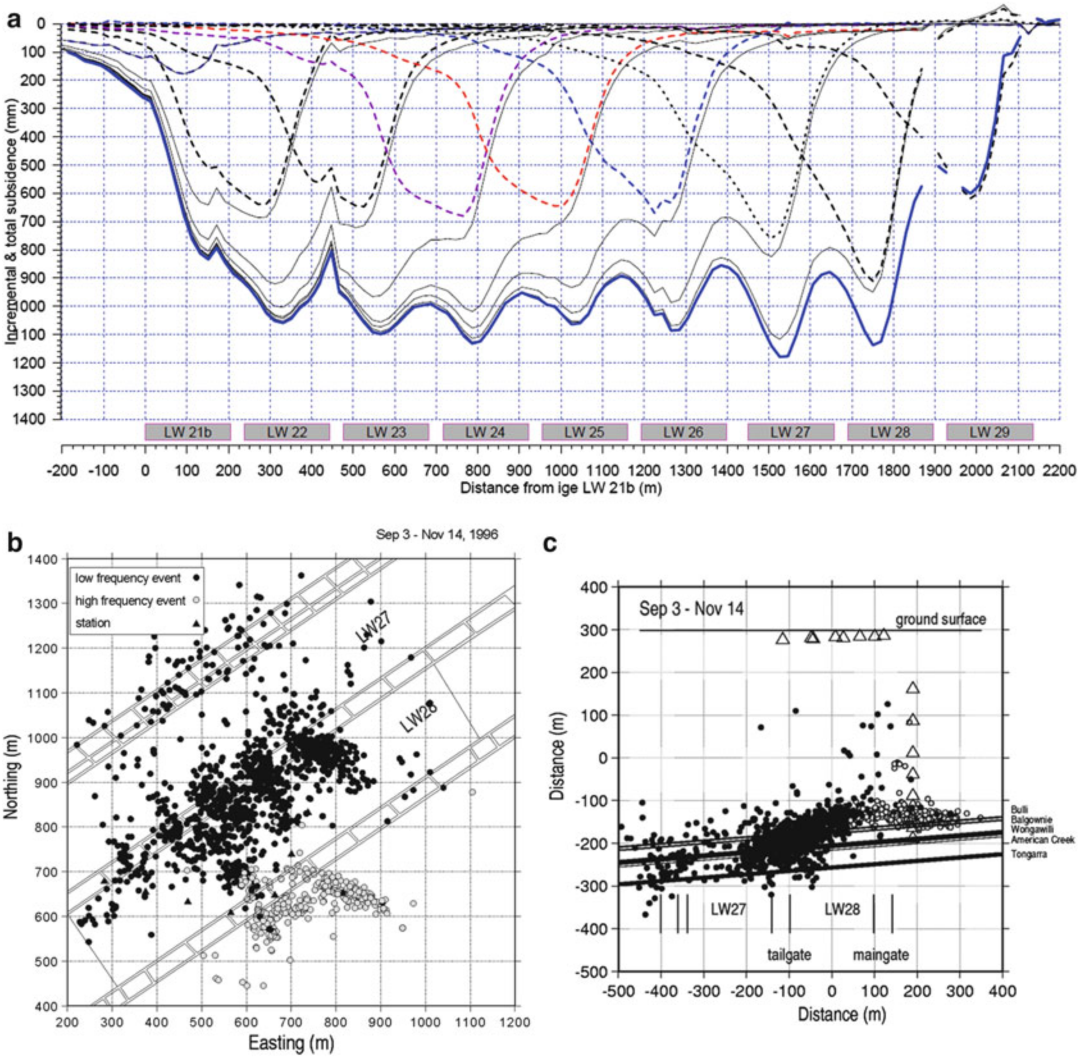


**Fig. 3.19** Surface subsidence profile and microseismic event location plots associated with extracting LW 28 at Appin Colliery. (**a**) Transverse incremental and total vertical surface displacement profiles ($H_{\text{solid rock}}$ = 410 to 455 m, $W/H_{\text{solid rock}}$ = 0.45 to 0.5), (**b**) Plan view, (**c**) Cumulative events in vertical plane ((**a**) Adapted from MSEC 2007; (**b**) and (**c**) after Hatherly and Luo 1999)

et al. 1998). Figure 3.19 shows how vertical surface displacement developed incrementally in these circumstances. Microseismic activity in the floor was biased towards the tailgate of the active longwall (Longwall 28) but also occurred beneath the tailgate of the previously extracted panel. Microseismic activity was also detected on a geological structure some 400 m ahead of the face. The microseismic monitoring identified cyclical caving behaviour, evident from Fig. 3.19b, with failure still tending to occur ahead of the face.

The profiles of vertical surface displacement and the biased nature of the microseismic activity at the Appin Colliery research site reflect that in subcritical panel width-to-depth situations, abutment load on interpanel pillars develops incrementally at moderate to large depths of mining as the stiffness of the superincumbent strata is reduced to zero during the course of extracting several panels. In other words, there is considerable interaction between adjacent workings. The abutment angle concept fails to encapsulate this behaviour.

The two preceding caving behaviours fall towards either end of the spectrums for mining depth and total extraction panel width-to-depth ratio. They contrast with that associated with panels of subcritical panel width-to-depth ratio in a layout where, because of shallow depth and/or relatively wide interpanel pillars, caving does not fully develop and is not significantly impacted by the extraction of subsequent panels in the series. That is, surface subsidence does not develop incrementally to any significant degree as the overall extent of extraction increases. Microseismic monitoring indicates that in these instances, failures are concentrated behind the face, rather than ahead of the face, and are predominantly tensile failures rather than shear failures (Frith and Creech 1997; Strawson and Moodie 2007).

The shape of curves showing panel width-to-depth ratio, W/H, plotted against vertical surface displacement for isolated panels, such as those shown in Fig. 3.14, reflect the fact that there is a transition rather than a step change between subcritical and supercritical caving behaviour. The magnitude of load transferred onto the abutments of an excavation is determined by the extent of subsurface caving and fracturing and, as reflected by Fig. 3.14, this may be more complex to calculate than simply invoking the concept of an abutment angle.

The increase in stiffness of the superincumbent strata with depth and the impact that this has on interpanel pillar load is reflected, for example, in experience with the two chain pillar design procedures, Analysis of Longwall Pillar Stability (ALPS) and Analysis of Longwall Tailgate Serviceability (ALTS). Abutment angles for these two design procedures vary across a broad range that includes 21° as reported by Mark (1992) for USA sites; 5.1°–24.7° deduced from stress measurements in Australia by Colwell (1998); and 11.5° at a depth of 530 m in Australia as reported by Moodie and Anderson (2011). In the case of Australian operations, it was reported that departure between the proposed ALPS pillar loading cycle and the monitored chain pillar loading behaviour was particularly evident for the deeper mines with low panel width-to-depth ratios and 'bridging' strata. Colwell (1998) also reported that some concern has been expressed in the USA that the chain pillar design methodology, ALPS, *'does not work very well'* at deep cover with particularly strong ground conditions.

Vandergrift and Conover (2010) report that it has been speculated that ALPS overestimates the load transferred to the gateroads under deeper cover. The authors advise that instrumentation data from a geotechnical program conducted at a depth of ~420 m to ~535 m appears to indicate that load-transfer to the gateroad pillars is less than previously assumed on the basis of ALPS and that this may help explain why gateroad pillars with relatively low calculated stability factors have performed adequately at that mine site. The researchers calculated abutment angles in the range of 3°–16°.

Similarly, the manner in which the transferred load is distributed within abutments is also a variable as it is influenced significantly by the stiffness and deformation properties of the immediate roof strata, coal seam and floor strata. For all other factors being constant, the location of the peak abutment stress moves further into the solid as the stiffnesses of the immediate roof, coal seam and floor strata decrease. Nevertheless, although abutment stress magnitude and distribution are variable, a number of empirical formulations have been developed that prescribe abutment stress distribution. Equations 3.10 and 3.11 are two which have found extensive application. Equation 3.10, proposed by Peng and Chiang (1984), defines the lateral extent of the side abutment zone, D, on the basis of depth of mining. Equation 3.11, proposed by Mark and Bieniawski (1987), defines the rate of decay of abutment stress in this side abutment zone.

$$D = 2.84\sqrt{3.3H}\,(m) \tag{3.10}$$

where

*D = lateral extent of side abutment zone (m)*

and

$$\sigma_{ax} = \frac{3L_s}{D^3} k\,(D - x)^2 \qquad (3.11)$$

where

$\sigma_{ax}$ = *abutment stress at distance x from the edge of the excavation*

$L_s$ = *total side abutment load based on abutment angle concept*

These types of relationships can be quite useful for making first pass assessments of abutment stress magnitudes and distributions. However, based on a consideration of applied mechanics principles, they cannot be expected to find universal application because they have no regard to the stiffness of the mining system. For example, as depth of mining increases, it is inevitable that panel width-to-depth ratio moves from being supercritical to being subcritical. This results in the formation of a bridge of superincumbent strata, the stiffness of which is not accounted for in the concept of abutment angle. Once a bridge is formed, the weight of the bridging strata (which determines abutment stress magnitude) increases in direct proportion to the thickness, $t_b$, of the bridge, while the stiffness of the bridging strata (which determines abutment stress distribution profile) increases in direct proportion to the cube of its thickness; that is, $(t_b)^3$. Subsequently, back-analysis of USA and Australian in situ stress data by Tulu and Heasley (2011) has confirmed that measured abutment stress magnitude and extent can deviate significantly from the values predicted by Eqs. 3.10 and 3.11 and that a range of additional parameters need to be incorporated into these formulations.

Pressure burst events provide further evidence of the variation in abutment loading with overall width of extraction at depth. For example, Agapiot and Goodrich (2000) reported on the extraction of 200–250 m wide longwall panels at a depth of 600 m in Utah, USA, in which pressure bursts only became significant after the third panel had retreated a distance of 460 m.

Abutment stress generation and distribution is more complex when considering behaviour in the corners of an extraction panel and requires a three-dimensional perspective. As the corners are approached, the in situ support to the undermined strata increases and the strike direction of fracturing associated with caving has to rotate through 90°. These changes can result in both a reduction in the dip of mining induced stress fractures and an increased bridging of strata across the corners of the excavation. The extent of these changes is dependent on the depth of mining, panel width-to-depth ratio, the nature of the immediate roof, and the width of the interpanel pillars.

Figures 3.12 and 3.20 illustrate some of these aspects. In the former case, the immediate roof strata was weak and bedded, with caving extending into the corner of the longwall panel at a relatively steep angle. In the latter case, the longwall panel had only recently commenced and full caving was yet to be established. The roof was stronger and not as bedded, resulting in caving developing at a much flatter angle and not extending into the corners of the panel.

Numerical modelling now offers significant benefits for quantifying stress magnitudes and distributions about total extraction panels. Examples include Fig. 3.21 and the numerical modelling studies reported by Salamon (1991a), Gale (2004), Peng (2008) and Esterhuizen et al. (2010b).

## 3.3.2 Strong Massive Strata

In geology, the term **massive strata** is used to describe a rock mass that has a paucity of well developed bedding planes. Although the term has been ascribed a variety of definitions in ground engineering (reference, for example, Wilson (1986), Frith et al. (1992b), MDG-1017 (1994), Singh (2000) and Gale (2009)), these generally capture this intent. In respect of caving behaviour over coal mine workings, the term has come to be associated with strata that have a capacity to span considerable distances and impede the development of subsurface subsidence and, consequently, goaf consolidation and vertical

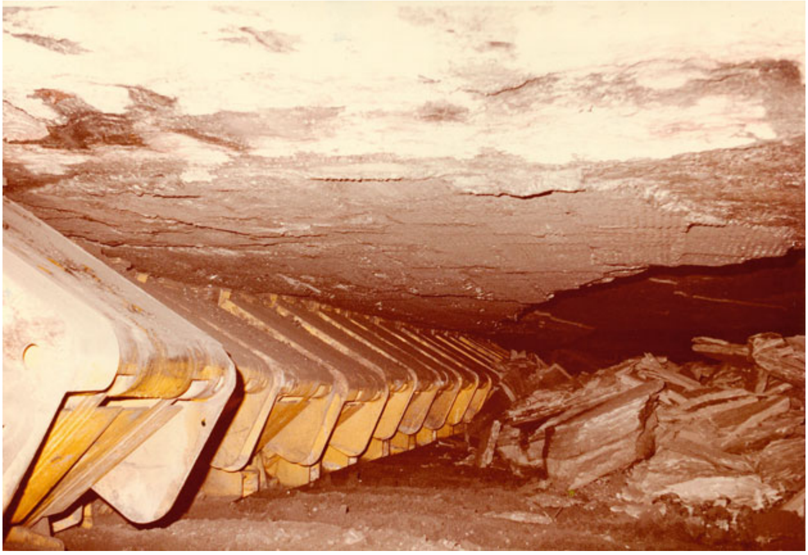


**Fig. 3.20** Caving behaviour of moderately strong, bedded roof strata at the gate-end of a longwall panel during the early stages of retreat

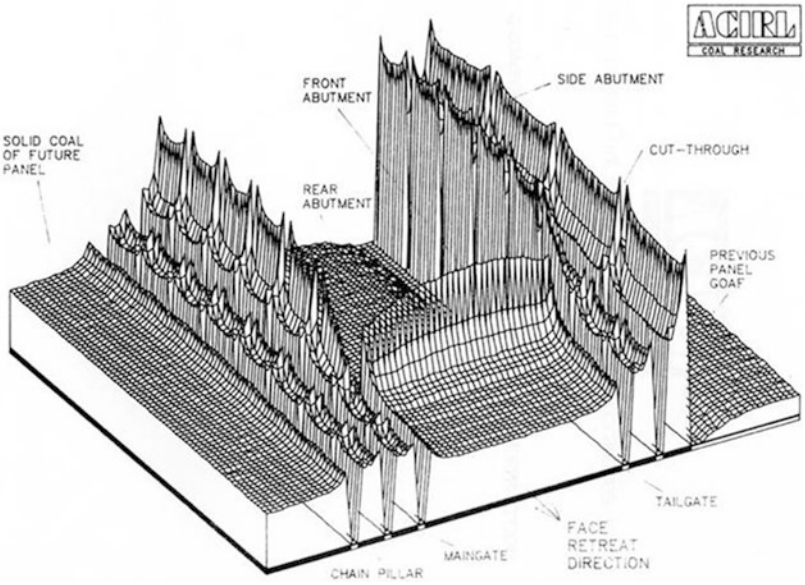

ACIRL
COAL RESEARCH
FRONT ABUTMENT
SIDE ABUTMENT
SOLID COAL OF FUTURE PANEL
CUT-THROUGH
REAR ABUTMENT
PREVIOUS PANEL GOAF
TAILGATE
FACE RETREAT DIRECTION
MAINGATE
CHAIN PILLAR

**Fig. 3.21** Stress magnitude and distribution about a longwall layout as determined by numerical modelling (After ACIRL 1984)

surface displacement. The spanning capacity of individual stratum increases with increase in their stiffness. In turn, stiffness increases with increasing stratum thickness, which is a reflection of how massive the stratum is, and with increasing elastic modulus, which has some imprecise correlation with strength. Hence, in the underground coal mining sector, the term 'massive' is often used loosely to refer to strata that are strong and have a capacity to span. These strata are not always massive in the strict geological meaning of the term. They most often comprise sandstones, conglomerates, limestones and dolerite sills.

Another legacy in the underground coal mining sector is to refer to strata as 'competent'. This term is not formally defined but to a coal miner, it generally implies that the rock mass is structurally stable and capable of spanning with minimal supplementary support in the given circumstances. This use of the term is scale dependent. For example, roof strata classified as ‘competent’ in a roadway might be considered ‘incompetent’ in the centre of a pillar extraction or longwall panel.

The reader needs to be alert to these terminological legacies and may sometimes need to deduce for themself what the terms ‘massive’ and ‘competent’ are intended to convey regarding rock mass properties. This text has endeavoured to avoid the use of the term ‘competent’ and to use the term ‘massive’ in accordance with its geological meaning. However, in some instances it has had to adopt the terms as used in published literature.

Dolerite sills have been implicated in a number of ground instabilities in first workings, pillar extraction and longwall mining, the most notable being the sudden collapse of Coalbrook Colliery in South Africa in 1960 (Moerdyk 1965) and the dynamic weighting of the longwall face at Churcha West Colliery in India in 1990 (Gupta and Ghose 1992). Their behaviour has been studied in detail in South Africa and provides a well researched point of reference for understanding the behaviour of massive strata. The sills are typically 30–70 m thick and are capable of bridging spans measured at their base of well in excess of 100 m, with associated vertical surface displacement of less than 300 mm (Galvin 1982). This is despite the presence of ubiquitous vertical, and to a lesser extent, horizontal cooling joints, as illustrated in Fig. 3.22. The lack of bedding planes in the material and its high intact strength are illustrated by the length of recovered core in Fig. 3.23. The base of this core had broken on a drilling induced fracture.

Salamon et al. (1972) applied a simple elastic thin plate model to estimate the span required to break a dolerite sill. The model was refined by Galvin (1981) to take account of the parting thickness between the coal seam and the base of the dolerite sill, the angle of caving of this parting, and the effect of overburden stiffness on surcharge load. This resulted in derivation of the formula given by Eq. 3.12 for calculating the minimum mining span required to induce failure of a dolerite sill.

$$W_m = \sqrt{1165t_d - 935\frac{t_d^2}{D_d}} + 2t_p cot\beta \qquad (3.12)$$

where

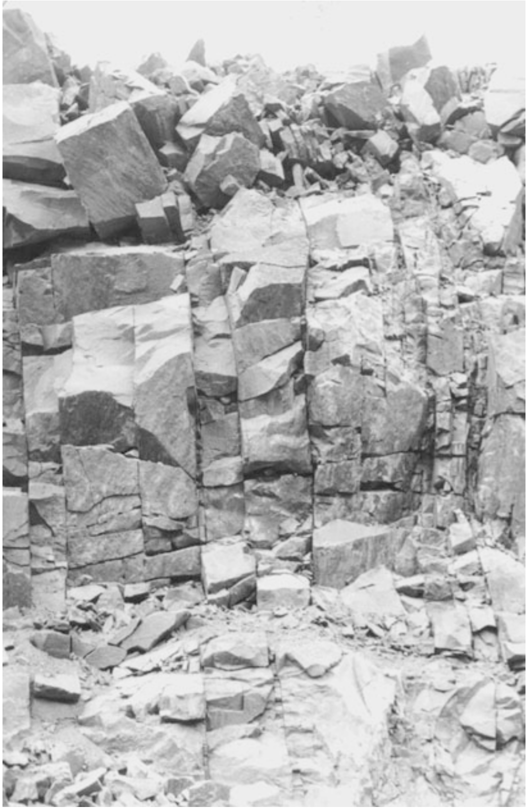


**Fig. 3.22** An example of the structural fabric of a dolerite sill after it has been impacted by blasting in a quarry (After Galvin 1982)

$W_m$ = *minimum span at mining horizon required to break sill*
$t_d$ = *thickness of dolerite sill*
$D_d$ = *depth to base of dolerite sill*
$t_p$ = *caving angle in degrees measured from the mining horizon*
$\beta$ = *caving angle in degrees measured from the mining horizon*

This formula has been applied extensively and with considerable success (Wagner 1994; Latilla and van Wijk 2003), albeit that it is a two-dimensional model applied to what is essentially a three-dimensional structure. Figure 3.24 shows how the maximum stress at the base of an elastic plate develops with face advance, L. Of practical importance is that the maximum stress acting in the base of a plate changes insignificantly once face advance exceeds 2.5 times the panel width. Hence, if failure of a

**Fig. 3.23** A length of core recovered from a dolerite sill illustrating the lack of bedding planes in the material and its high intact strength (After Galvin 1981)

massive stratum has not occurred by that stage and geological conditions do not deteriorate, it is unlikely to occur with further face advance.

In addition to generating elevated abutment stresses, the presence of an unfailed massive bed in the superincumbent strata of an extraction panel can result in an increase in the lateral extent of elevated abutment stresses. This response is not captured in empirical formulae for abutment stress distribution that are based only on depth of mining, such as that defined by Eq. 3.10. Both effects can cause serious stability problems at the mining face and in flanking roadways, panel pillars and interpanel pillars in the lead up to the initial failure of the massive strata and, if it does not break, for the life of the panel. One control implemented in these circumstances in total extraction operations in South Africa and Australia is to reduce panel width, typically from the order of 200 m back to 100–130 m (Henderson 1980; Galvin et al. 1982; Beukes 1989; Frith and Creech 1997) and to leave substantial pillars between panels.

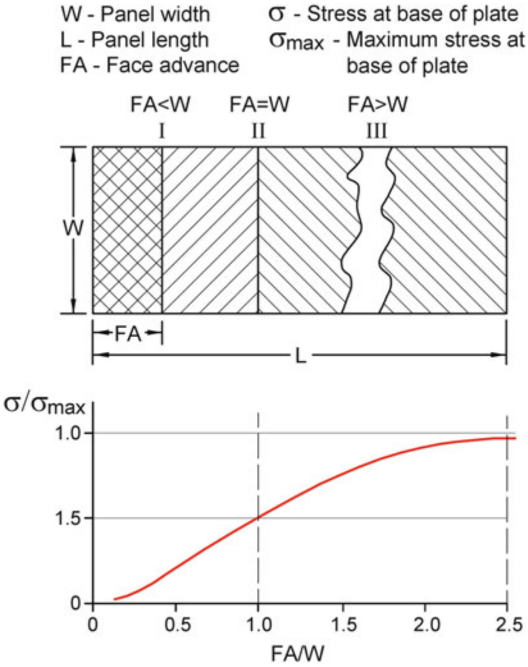


**Fig. 3.24** Development of maximum stress at the base of an elastic plate as a function of the ratio of face advance to panel width (Adapted from Galvin et al. 1981)

Failure of a massive stratum effectively unclamps only one of the four edges of the plate and so the stratum still has a capacity to cantilever off the face line for a considerable distance into the goaf before failing, thereby generating high abutment pressures on a cyclical basis. The bridging of a massive stratum also results in the caving and subsidence process being interrupted, causing a cavity to form beneath the massive stratum and, thus, discontinuous subsidence. If the bridging stratum is within the caving height, $h_c$, caving will cease before the goaf is choked due to bulking. If it is higher in the roof, subsidence of the underlying strata will result in a gap beneath the massive stratum, as illustrated in Fig. 3.25. Depending on mining height and parting thickness, this gap could range from hundreds of millimetres up to several metres. The formation of a gap can give rise to windblast, gas inrush and water inflow hazards in the event of failure of the massive stratum.

On the basis of elastic plate theory, once flexural fracturing of a massive stratum is initiated, it should result in an increase in bending stress in the remainder of the stratum, thus causing

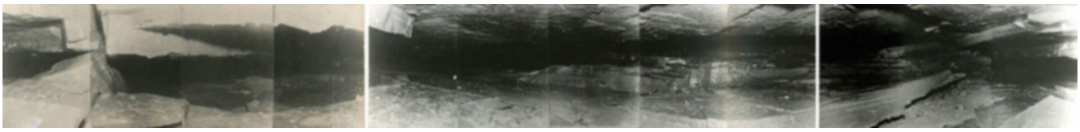


**Fig. 3.25** A 360° view of a 300–700 mm wide gap within a dolerite sill associated with discontinuous subsidence (Courtesy of Professor Miklos Salamon)

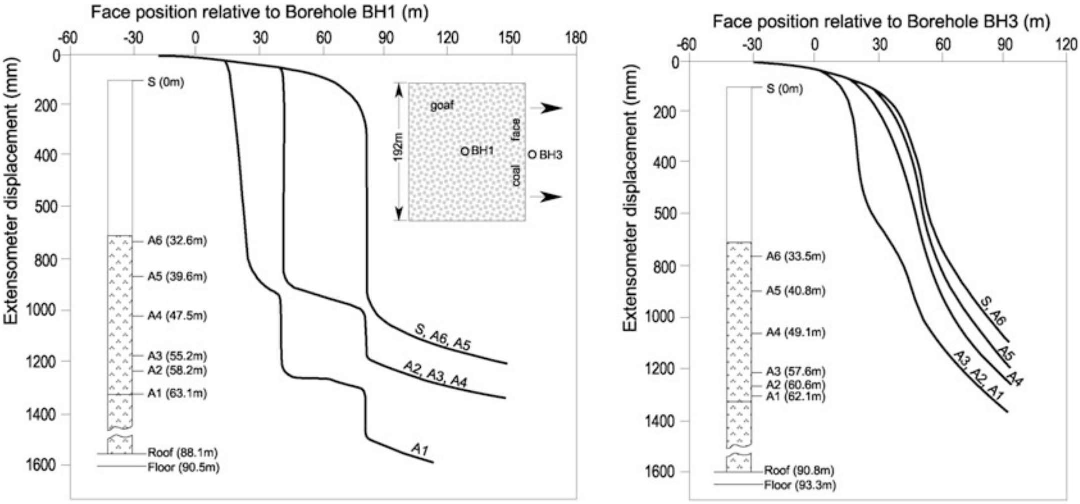


**Fig. 3.26** Graphs showing progressive step-failure of a dolerite sill as recorded using surface-to-seam borehole extensometers (Adapted from Salamon et al. 1972)

fracturing to propagate rapidly through the rock mass. However, field measurements in South Africa have revealed that this failure process may develop in a number of distinct steps that are dependent on further increases in the minimum panel span (Galvin 1983). Field observations indicate that similar behaviour can occur in massive conglomerate/sandstone stratum in Australia. Gupta and Ghose (1992) concluded that the dynamic periodic weighting accident at Churcha West Colliery in India, which destroyed 23 longwall hydraulic face supports, was associated with failure of the lower 19–30 m of a 125 m thick dolerite sill, some 85 m above the working horizon.

Figure 3.26 shows caving behaviour in a dolerite sill above a longwall panel in South Africa, as recorded using multipoint surface-to-seam extensometers. Subsidence terminated initially at the base of the sill and then progressed through it in two distinct steps. Immediately prior to full collapse, the longwall panel was bridged by a beam of dolerite that was somewhere between 7.6 and 15.5 m thick and supporting its own weight and that of 36 m of overburden. It has been suggested that the behaviour may be associated with the redistribution of horizontal stress as failure progresses through the massive strata, with the increased lateral stress improving the structural stability of the remaining jointed material (Galvin 1983). With subsequent advances in the rock mechanics knowledge base, it is possible that it may also be attributable to the formation of linear arches (voussoir beams).

Of particular significance in Fig. 3.26 is the small amount of surface subsidence up to the onset of the final collapse and the rate at which subsidence developed. Similar behaviour is associated with total extraction beneath massive conglomerate/sandstone strata in the Newcastle Coalfield, NSW, Australia. This behaviour illustrates how surface subsidence measurements

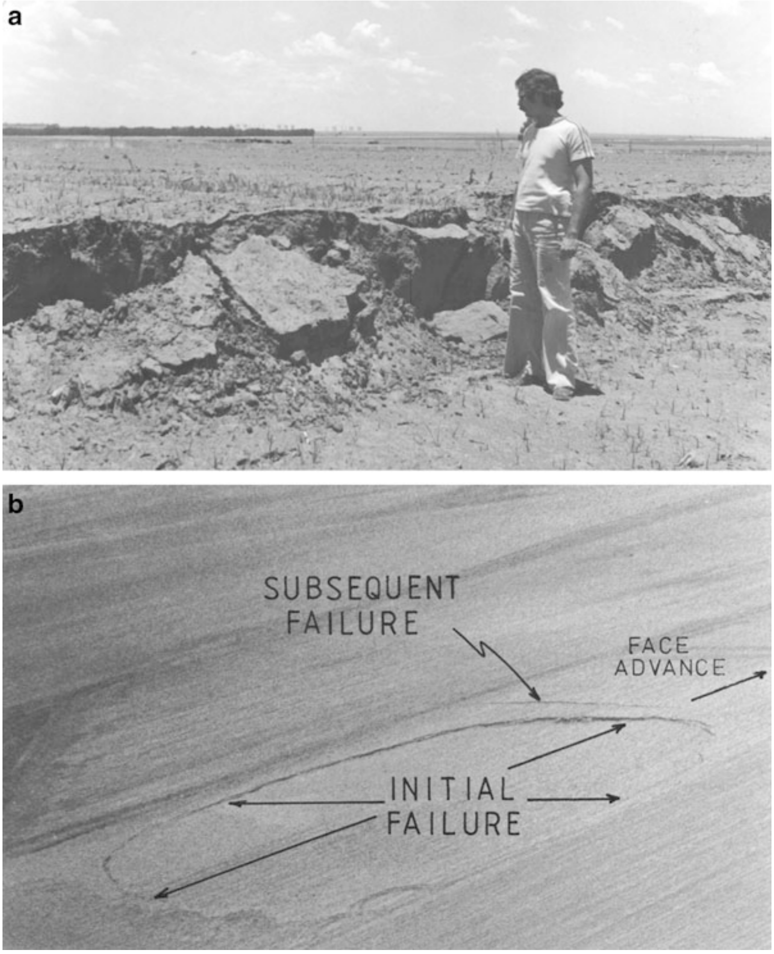


**Fig. 3.27** Apparent abutment shear failure above a massive dolerite sill overlying a longwall panel. (**a**) Steep step in surface profile at the perimeter of the subsided plug. (**b**) Overall view of subsided plug (After Galvin 1981)

can be a poor indicator of the overall state of caving and stability of a massive strata. Therefore, it is advisable in sensitive or critical situations to monitor subsurface behaviour with extensometry. Field observations also reveal that, on occasions, the final failure of massive strata may be associated with shear failure at the panel abutments, shown in Fig. 3.27, which is not inconsistent with the plug failure model of Galvin (1982b) or with the failure of a linear arch.

Three potentially serious hazards associated with total extraction mining beneath massive strata are excessive abutment stress throughout the operating life of a panel, periodic weighting, and windblast. From an operational perspective, it is desirable to make a panel sufficiently wide to induce failure of massive strata as soon as possible after the commencement of mining. Alternatively, the panel span needs to be restricted so that caving does not occur and abutment stresses are not excessive. A situation to be avoided is where the panel span is only marginally less than the critical span, such that mining operations are subjected to high abutment stress for the operational life of a panel and prone to a small change in geology triggering the collapse of a large area of strata within the goaf. This latter situation creates the potential for the ingress of flammable and noxious gases into the workplace and for windblast.

If panel span is insufficient to induce full caving of the superincumbent strata, then careful consideration has to be given to the separation distance between panels. This is to avoid both exposing subsequent operations to high abutment stress and exposing personnel to an inrush or windblast caused by the collapse of a large area of standing goaf in the previously extracted panel.

In some situations, a massive rock mass may fail in stages in a rapid but not uncontrolled manner. Hardman (1971) reported that of a total movement of 700 mm in the lower half of a dolerite sill when it failed, 530 mm occurred within a period of about 1 min. Gupta and Ghose (1992) reported that 1500 mm of leg closure was recorded in less than 1 h in the incident attributed to the catastrophic failure of the base of a dolerite sill in India. Total extraction beneath massive conglomerate strata in the Newcastle Coalfield of NSW is well known for resulting in sudden and dynamic failures of the conglomerate that generate severe windblasts (see Sect. 11.1) It is noteworthy that the consequences of failure of massive dolerite sills in South Africa have not been as severe as the failure of massive strata in Australia. This appears to be due to adherence in South Africa of the advice of Salamon and Oravecz (1976). They advised that it would be prudent to provide a protective cushioning to a failing dolerite sill by restricting total extraction to areas where the normal shale/sandstone parting between the dolerite and the seam was not less than 8–10 times the mining height.

### 3.3.3 Span Design

So-called ‘first working’ mining systems in coal mining are based on bord and pillar mining in which coal extraction is confined to relatively narrow roadways that define coal pillars whose purpose is to maintain the integrity of the superincumbent strata. As the depth of mining increases, percentage extraction rapidly drops in these mining systems because of the need to leave larger pillars to support the increased weight of the overburden. Hence, subject to safety and environmental considerations, mining usually reverts to some form of secondary extraction in which coal pillars are subsequently extracted to form wide excavations. These secondary extraction mining methods are loosely referred to as ‘total extraction’ mining methods and are dominated by pillar extraction (discussed in Chap. 8) and longwall mining (discussed in Chap. 9).

The determination of panel span in total extraction situations is a site-specific matter since it is a function, amongst other things, of the composition, thickness, depth and relative position of the various stratum that make up the overburden. Design can be aided considerably by prior operational experience in the given or similar conditions.

The formation of a secondary extraction panel can give rise to high abutment stresses. While considerable research has been undertaken into quantifying abutment stress distributions about longwall entries, or gateroads, and developing design and support procedures for controlling the impacts of this abutment stress on these roadways, the same is not the case for managing impacts on the mining face. Ideally, mine design should result in either:

- an excavation span (width) that is sufficiently wide so as to induce full caving and subsidence of the overburden very soon after the commencement of secondary extraction in order to maximise relief from abutment stress; or
- an excavation span that is sufficiently narrow so as to restrict abutment stress.

In practice, this can be a complex design exercise. Not only is face abutment stress indeterminate because it is a function of the stiffness of the mining horizon and of the surrounding strata, but also in secondary extraction situations the stiffness of the surrounding strata is also governed by the caving and subsidence behaviour of the overburden and the reconsolidation characteristics of the goaf. The situation becomes more challenging as depth increases because face abutment stress is determined increasingly by the degree of interaction between panels, as reflected in the variation in the profiles of vertical surface displacement shown in Figs. 3.14, 3.15, 3.16, 3.17, 3.18, and 3.19. Neither ideal span designs may be achievable at depth, simply because they result in spans that are either too small to be economically viable or too large to be practically achievable.

In addition to abutment stress, span design may need to take account of a range of other factors, three of the more important being windblast (see Sect. 11.1), subsurface subsidence impacts (see Sect. 10.3), and surface subsidence impacts (see Sect. 10.4). Often, these are the primary determinants of panel span, with their control usually resulting in panel span having to be restricted to a subcritical width. Many serious incidents have been associated with this approach when panel span has been only marginally less than that required to induce full caving. In these situations, the extraction line may be subjected to high abutment stress throughout the life of the panel; ground control can be very susceptible to small changes in geology; rib spall may be severe; and localised caving may occur on an irregular basis. Hence, strata behaviour is inconsistent and unpredictable. These situations serve as examples of where the controls introduced to mitigate one risk can introduce new and sometimes higher risks. This is why it is essential that risk management controls are also risk assessed in their own right (see Chap. 12).

A number of different methodologies are utilised in practice to select panel span. None are universally applicable and all have strengths and weaknesses. These methodologies include:

- Experiential, usually developed out of trial and error. This approach is epitomised in pillar extraction, where there is a multitude of variations in extraction technique and panel span and strong regional preferences for certain variants and mining dimensions.
- Empirical. There is a variety of empirical approaches, with the oldest and most extensively applied being based on observations of vertical surface displacement above isolated total extraction panels. These observations are often presented in the form of dimensionless plots, such as that shown in Fig. 3.14, by normalising panel span, or width, with respect to depth and plotting this ratio against vertical displacement normalised with respect to mining height. The variety of curves shown in Fig. 3.14 mainly reflects the different site-specific geomechanical conditions. The curves display the same generic trait of vertical surface displacement increasing gradually with increase in excavation width-to-depth ratio, W/H, then accelerating through a transition zone, before reaching a peak value at a width-to-depth ratio of around 1–1.2.

These type of curves provide insight into the state of caving about a total extraction panel and can assist in deducing the state of abutment stress. However, limitations are associated with them, including that they can mask discontinuous subsidence situations and do not reflect increasing interaction between panels as depth of mining increases for a given panel width-to-depth ratio, W/H. This latter limitation can be overcome to some extent by computing the increment in vertical displacement associated with the extraction of each panel. Figures 3.16 and 3.19 are examples of this approach. Plots of incremental subsidence give better insight in the development of abutment stress but deductions still have to be made regarding abutment stress magnitude and distribution.

Empirical formula such as those defined by Eqs. 3.10 and 3.11 and variants of these developed for pillar extraction situations (such as the Analysis of Retreat Mining Pillar Stability Methodology (ARMPS) (Mark et al. 2011)), provide an alternative means for estimating abutment stress magnitude and distribution that do not depend on local knowledge or experience. Nevertheless, these types of approaches are still constrained by an inability to fully consider the stiffness of the loading system.

It has been suggested by some researchers that the mining rock mass rating system (MRMR) as developed and applied by Laubscher (1994) to predict the caveability of superincumbent strata could find application in coal mining situations. Although a number of coal mine sites are included in the database that underpins this rock mass classification system and its subsequent iterations, this approach has found very limited application in the coal sector to date.

Empirical approaches to panel design predominate in situations where the undermining of strong and massive stratum can give rise to the risk of windblast. A number of mines in the Lake Macquarie region of Australia developed their own panel span criteria for extracting coal pillars directly beneath a massive and strong sandstone/conglomerate roof strata in the Great Northern Seam (personal experience). Frith and Creech (1997) developed an empirical relationship for designing panel width to mitigate face instability when longwall mining directly under similar strata in the northern region of Lake Macquarie. While the procedure proved successful for preventing very large rib falls that then gave rise to large roof falls, the associated reduction in panel width resulted in violent windblasts that, arguably, presented a greater risk to personnel than the risks associated with falls of rib and roof that the procedure was intended to mitigate.

- Semi-empirical. Generally, this approach is based on simple analytical models that have been refined and calibrated using empirical data. The Galvin dolerite span formula, given by Eq. 3.12 is one such approach. This formula has been in use for over 30 years and has served the South African mining industry well in determining the mining spans required to induce full caving of dolerite sills. Nevertheless, reliance still has to be placed on operational experience in assessing if tolerable abutment stress conditions are likely to be associated with the calculated critical span. Because the formulation was derived on a database specific to the behaviour of dolerite sills in South Africa and is not mechanistically rigorous, it does not find universal application, although it has provided good approximations in some situations associated with strong and stiff sedimentary strata.
- Analytical. Linear arch theory (Sect. 2.8.8) finds application in both civil and mining engineering practice in calculating spanning capacity (reference, for example, Beer and Meek 1982; Wold and Pala 1986; Pells and Best 1991; Nomikos et al. 2002; Seedsman 2004; Brady and Brown 2006). However, these applications are primarily focussed on assessing spanning capacity and vertical surface displacement and do not provide an assessment of abutment stress magnitude and distribution in and about a mining face.
- Numerical. In theory, numerical modelling offers the significant benefit of being able to take account of the stiffness of the loading system and, therefore, to produce abutment stress magnitude and distribution profiles around the full perimeter of an excavation, as illustrated in Fig. 3.21. However, in practice, considerable uncertainty can be associated with the accuracy of input data, especially in regard to caving and goafing characteristics. Limitations are associated with three-dimensional numerical modelling, especially when attempting to simulate some pillar extraction layouts.

In summary, surface subsidence observations provide insight into the state of abutment stress and provide a sound basis for testing the validity of numerical models. Semi-empirical and analytical models can provide reasonably accurate estimates of the span required to induce full caving and subsidence if calibrated to site-specific data. Some also produce abutment stress magnitude and distribution profiles but care is required when these models are applied to situations that fall outside the site-specific conditions for which they were derived. Numerical models can be very helpful for quantifying abutment stress magnitudes and distributions as a basis for selecting mining span. However, they may also be unreliable and, therefore, outcomes should be used as an aid and supported by parametric and sensitivity analysis, rather than being accepted blindly as accurate simulations of reality. Irrespective of the desktop approach taken to design, local operational experience is generally invaluable when determining mining span.

## 3.4 Elevated Horizontal Stress

Underground coal mining often takes place in environments where the major principal stress is horizontal. The adverse impacts of high horizontal stress on weak, bedded and laminated roadway roof and floor strata can be avoided by orientating roadways parallel to the direction of the major horizontal principal stress. However, this is rarely possible for many reasons. Stress direction can change across the mining lease, in the vicinity of geological structures, and beneath topographic highs. In any case, roadways still need to be connected at regular intervals by cut-throughs. Practical considerations such as lease boundaries, surface constraints, and seam dip also influence roadway direction. Roadway direction relative to seam dip, for example, has implications for water management, ventilation control, and equipment stability. In some cases, the minor horizontal principal stress may also be of sufficient magnitude to adversely impact roadway stability. Hence, compromises have to be made and it is almost inevitable that at some stage in the mining cycle, roadways will have to be developed at an angle to the major horizontal principal stress direction.

Figure 3.28 is an example of a streamline model that has found extensive application in elevated horizontal stress situations in coal mining to account for poor ground conditions biased to one side of the mining face, with this side varying with the direction of mining. It shows conceptually and correctly how in-seam horizontal stress is redistributed ahead of, under and over the face of a roadway that is advancing at an acute angle to the direction of the major horizontal stress. The poorer roof conditions, guttering and floor heave encountered on the ‘leading corner’ of roadways in elevated horizontal stress situations have often been attributed in the past to in-seam stress redistribution being concentrated about that point.

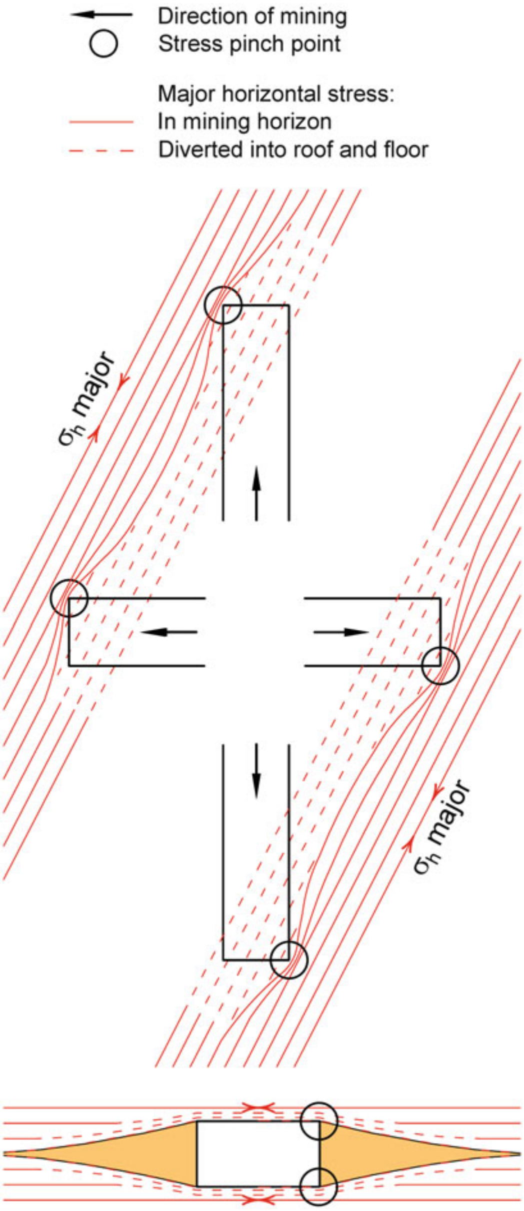


**Fig. 3.28** An example of a widely adopted and now highly questionable streamline model to account for biased adverse roof and floor conditions in a high horizontal stress field in an underground coal mining environment

Although the model correctly predicts the site of most impact, attributing the impact to the redirection of in-seam stress is questionable in the case of underground coal mining. This is because it is generally the case that coal seams are not subjected to high horizontal stress (see Sect. 2.6.7). The in-seam stress that is redistributed into the roof and floor strata may aggravate a state of instability but, because of its low magnitude, this stress is very unlikely to be the primary cause of the poor roof and floor conditions.

The biased face conditions in coal mining are most likely attributable to a reduction in confinement to the immediate roof and floor strata. As the roadway is advanced, the normal stresses acting on the immediate roof and floor strata are removed, whereas the horizontal stresses acting in the immediate roof and floor strata remain high and may increase slightly due to the redistribution of the in-seam stress. This changed state of stress creates a high propensity for buckling and flexural (bending) failures of roof and floor strata, especially in bedded or thinly laminated strata. Softening and failure of roof and floor strata around the leading corner of a roadway effectively creates a stress relief slot (see Sect. 5.2.3) that then provides a degree of horizontal stress relief to the immediate roof and floor strata over the remaining width of the roadway.

In the case of the roof strata, the onset of failure and the extent of roof damage depend to a large extent on the magnitude of the horizontal stress field, the structural and strength characteristics of the immediate roof strata, the minimum distance that support and reinforcement can be installed from the face and the timing of the installation of these ground control measures. In very weak strata and/or high horizontal stress situations, the roof may gutter close to or at the affected face corner as excavation is occurring. Ground control largely depends on minimising the span between the face and the last row of installed ground support and on the effectiveness of this support system. It stands to reason, consistent with field experience, that there are benefits in these situations in installing ground support as soon and as close as possible to the face. This is to promote beam building; to mobilise the residual strength of failed strata; and to prevent unravelling and falls of ground, with additional reinforcement and support elements being installed on the affected side of the roadway in an endeavour to restore some degree of confinement.

In the seam horizon itself (ribsides and face), elevated horizontal and vertical stresses exist immediately ahead of the leading corner. As a result of roadway advance, the horizontal stresses acting normal to the ribsides and face will become zero whereas the tangential stresses acting in the ribside and face will remain relatively unchanged. The vertical stresses in the rib side and the face also do not change significantly as a result of roadway advance. Hence, these in-seam stress changes are not as pronounced as those that impact the immediate roof and floor strata. Against this background, it is suggested that the impact of high horizontal stress in the roof and floor strata is better conceptualised as shown in Fig. 3.29.

The problem is truly three-dimensional. Three of the roof beam abutments are clamped but the fourth has indeterminate end constraints. At and around the corners of the roadway, the principal stresses are not aligned with the roadway span or with the direction of roadway advance. Amongst other things, this causes shear stresses to act in these directions and between strata. These factors have important implications when attempting to calculate the stresses (and equivalent forces) that lead to roof and floor bending and buckling. Analysis of this environment and associated ground support and reinforcement requirements falls well outside the scope of simple beam theory.

It is essential when mapping roadway conditions for the purpose of interpreting stress direction that, as evident from Fig. 3.29, careful note is made of the direction from which the roadway was mined. When both sides of a roadway show signs of guttering, it is likely that either the roadway orientation is within 30° of being normal to the major horizontal stress direction or the minor horizontal stress is also elevated. Once a roadway is within 30° of being

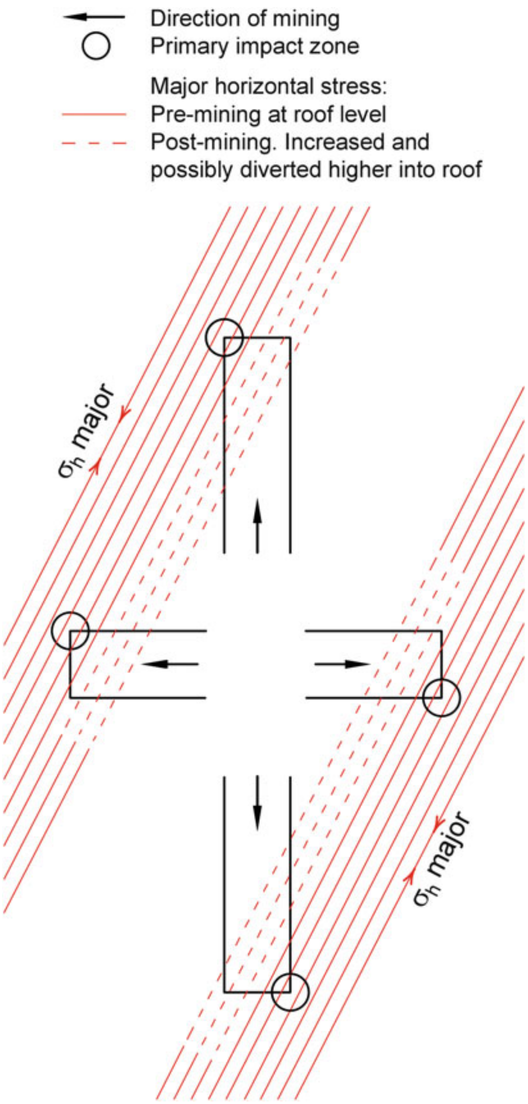


**Fig. 3.29** A preferable model for determining the location of biased adverse roof and floor conditions in a high lateral stress field

parallel to the direction of the major horizontal stress, impacts tend to reduce rapidly with further decrease in this angle. Mine design and support measures for mitigating the hazards associated with elevated horizontal stress are discussed in more detail in Chaps. 7, 8, 9 and 10.

## 3.5 Shallow Mining

### 3.5.1 Principles

Analytical and numerical modelling confirm that at great depth, most of the rock surrounding a wide tabular excavation is subjected to compressive stresses. This means that the regional behaviour of the excavation is influenced little by geological discontinuities and is controlled mostly by the inter-relationship between the prevailing stresses and the mechanical properties of the rock, making it relatively easy to predict rock behaviour (Salamon 1983).

A similar situation can exist at shallow depth when the horizontal stress around an excavation is typically two or more times the vertical stress; that is, $k \geq 2$. However, the situation changes significantly as this stress ratio reduces and can present a very serious risk of unpredictable behaviour once the stress ratio is less than one. Even in a high horizontal to vertical primitive stress environment, these situations can arise at shallow depth in total extraction situations. This is because caving and subsidence disrupt the transmission of horizontal stress in the superincumbent strata, resulting in adjacent mining panels being located in a horizontally destressed environment.

A number of Australian and USA longwall operations have operated at depths as shallow as 18–50 m (Holt 1989; Frith et al. 1992a; Butcher and Kirsten 1999) while pillar extraction has occurred at depths at least as low as 30 m in South Africa (Schumann 1982) and 20 m in Australia (Enright 1995). As depth of mining decreases below about 100 m, and especially below 50 m, excavation behaviour becomes increasingly sensitive to small changes in geology, dimensions of the mine workings and field stress and, therefore, more unpredictable. In particular, as depth decreases:

- Geological features such as joints, faults and dykes are likely to be more weathered, open and continuous between the mine workings and

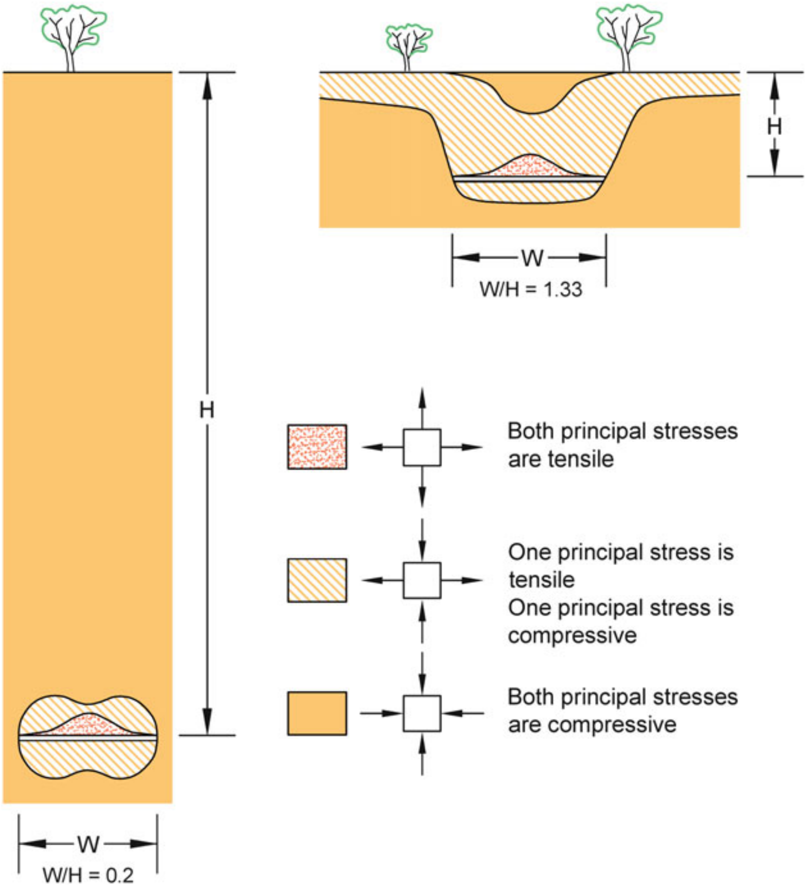

H
W
W/H = 0.2
H
W
W/H = 1.33
Both principal stresses are tensile
One principal stress is tensile
One principal stress is compressive
Both principal stresses are compressive

**Fig. 3.30** Comparison of tensile zones above and beneath isolated total extraction panels at shallow depth and great depth for k = 0.5 (Modified from Galvin et al. 1982, after Salamon 1974)

the surface. This increases the potential for shear failure, which is elevated in wet weather due to the reduction in effective normal stress across potential sliding surfaces and, in some circumstances, lubrication of these surfaces (such as in some clay infill situations).

- The impact on overburden stiffness of unconsolidated surface deposits is proportionately greater. This impact cannot be ignored at shallow depths, with the effective depth of (solid rock) cover having to be reduced by a corresponding amount and loaded with a corresponding deadweight surcharge. In total extraction mining at shallow depth, a small increase in the thickness of unconsolidated overburden or in the overall span of the panel can cause a large increase in the effective panel width-to-depth ratio, resulting in both a significant reduction in the overburden load transferred to the panel abutments and a significant increase in the likelihood that any panel collapse will be uncontrolled.

Elastic continuum numerical models provide insight into overburden behaviour at shallow depth, albeit that such models may not be representative of the discontinuous nature of many rock masses at shallow depth. Figure 3.30 shows the zones of principal stress about a shallow panel and a deep panel of the same width predicted in this manner for a stress field in which the horizontal to vertical stress ratio is 0.5 (Salamon 1974). The figure highlights that the effect of a free surface on the stress distribution around an isolated panel cannot be ignored once panel width-to-depth ratio exceeds 0.4. In the shallow case:

- the vertical extent of the zone where both principal stresses are tensile has increased;

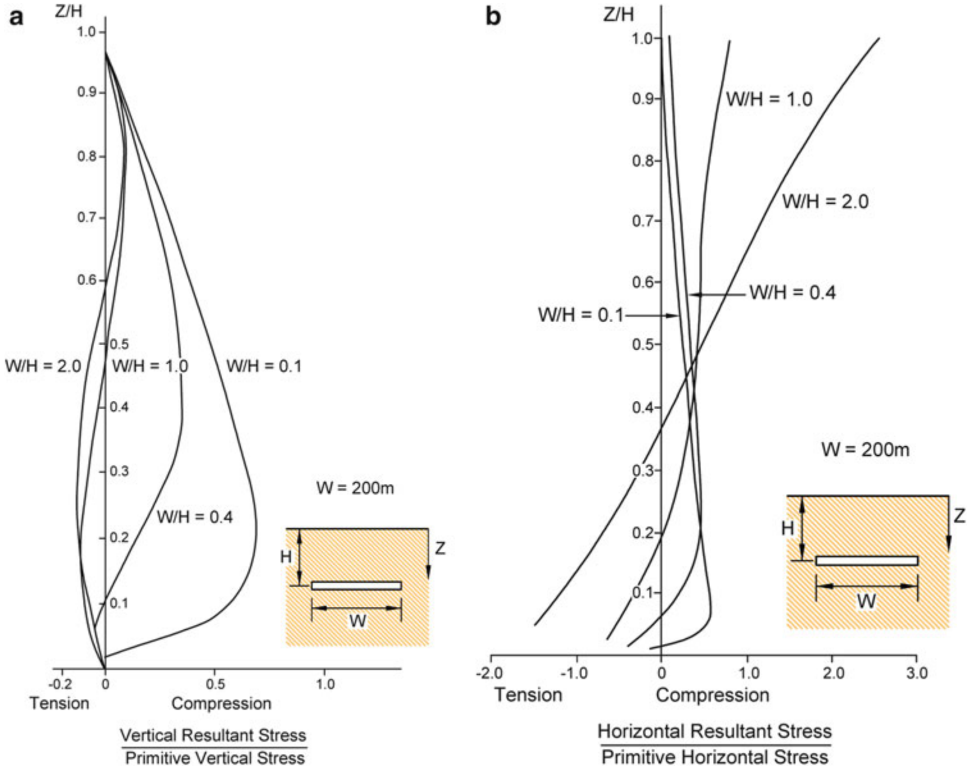


**Fig. 3.31** Normalised total vertical and horizontal stress above the centre of an isolated panel for various total extraction panel width-to-depth ratios, W/H, for k = 0.5. (**a**) Normalised total vertical stress. (**b**) Normalised total horizontal stress (Modified from Galvin et al. 1982 after Salamon 1974)

- the zone where one of the two principal stresses is tensile extends right through to surface and outside the vertical projection of the panel; and
- an isolated zone where both principal stresses are compressive exists above the panel.

The total vertical stress (normalised with respect to primitive vertical stress) above the centre line of a panel for various values of panel width-to-depth ratio, W/H, for the case of $k = 0.5$ is shown in Fig. 3.31a. As depth of mining decreases, an increasingly larger proportion of the strata between the roof of the excavation and the surface is subjected to tensile stresses. In fact, from the stage when the span is equal to the depth, the vertical stresses are tensile in the lower half of the overburden. Similarly, the proportion of the roof strata where the horizontal stresses are tensile increases with increasing panel width-to-depth ratio (Fig. 3.31b).

The following important conclusions can be drawn from Fig. 3.31:

- The low tensile strength of rock and the presence of many structural weaknesses in the rock mass, such as bedding and joint planes, become significant features in the presence of extensive zones of tensile stress. Caving of the roof strata right through to the surface is a distinct possibility, particularly if the overburden comprises weak and friable rock.
- Only massive, strong rock beds are likely to remain unfractured once panel width exceeds twice depth ($W > 2H$).
- The tendency of the roof strata to cave through to the surface increases with increasing values of panel width-to-depth ratio.
- The existence of a continuous zone of tensile stresses from the surface through to the underground workings creates potential paths for the inflow of surface water.

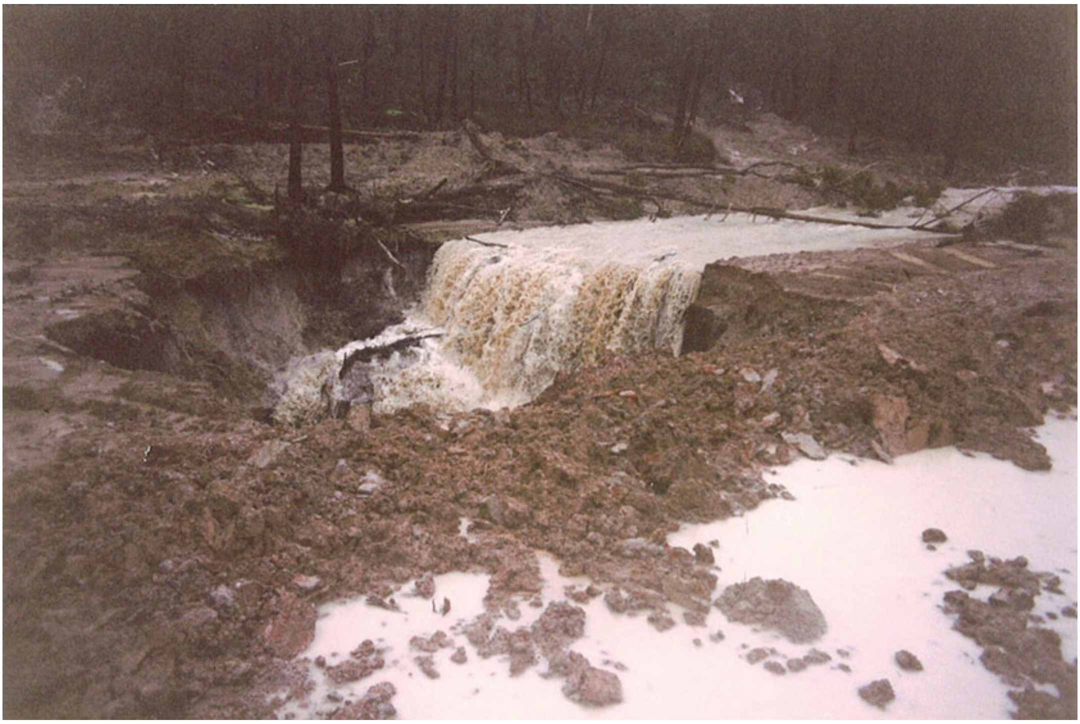


**Fig. 3.32** An example of a watercourse breaking through into shallow pillar extraction workings during a rainfall event

- Zones of high tensile strain and zones of high compressive strain are to be expected on the surface as a result of mining wide panels at shallow depth.

A second type of caving failure at shallow depth is that of **chimney caving** or **sinkhole** formation. In coal mining, these terms refer to failures of the roof strata over bords, particularly over intersections, that extend through to the surface. Sinkholes can increase the risk of spontaneous combustion of coal within the mine workings (because they permit the ingress of oxygen) and can present a risk to safety and the environment. The failures may develop as a plug but more often as falls of ground that progressively extend through to the surface. In the later case, bord width, mining height, bulking factor, depth, and the composition, competence and angle of repose of the superincumbent strata influence the development of a sinkhole. Canbulat (2003) has provided an analytical model for evaluating these situations. Brady and Brown (2006) present a model based on limit equilibrium that finds application to shear failure in strong rock devoid of discontinuities. This type of behaviour is discussed in more detail in Chap. 10.

### 3.5.2 Practice

The ingress of rainfall into shallow total extraction workings is a common problem in underground coal mining due to the enhanced conductivity of the fractured overburden. With few exceptions, total extraction workings at shallow depth also result in wide surface cracks, sufficient to cause loss of water from natural and man-made storages and total loss of surface flow in watercourses. Surface water is diverted into the mine workings and/or downstream through sub-surface fracture networks

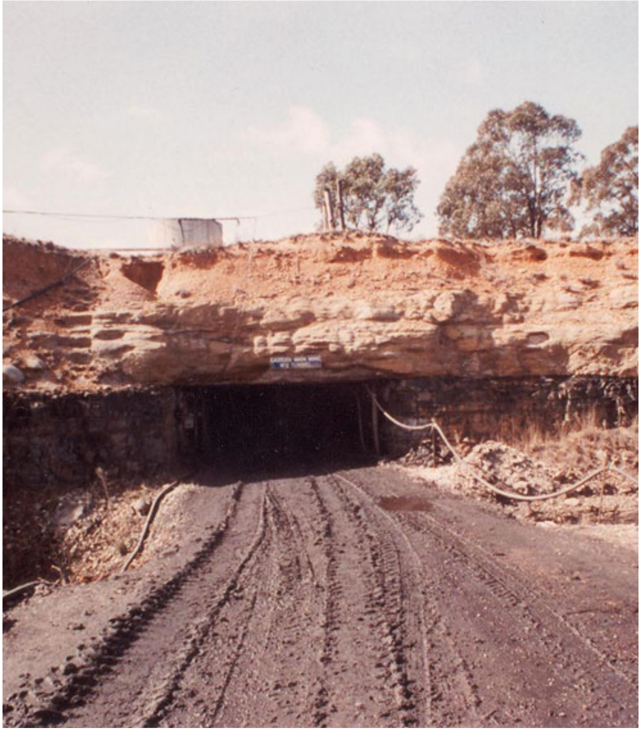


**Fig. 3.33** Entrance to a shallow mine showing that almost 50 % of the overburden comprises a deadweight surcharge, hence resulting in roadways having a large effective panel width-to-depth ratio (W/H >3)

(see Sect. 10.3). Often, open fractures are remediated by ploughing, ripping and dozing. However, this process does not guarantee that fractures are sealed to any significant depth and it is not unknown for ephemeral water courses to break through into mine workings during flood events, such as shown in Fig. 3.32. A more permanent solution when fracturing is not extreme involves sealing the fractures with clay. In some cases, such as where oxygen ingress is to be prevented in order to control spontaneous combustion, this may require the placement of a clay blanket which, obviously, can have environmental implications.

In some shallow situations, a high panel width-to-depth ratio can be associated with a mine roadway, as in Fig. 3.33. When the thickness of the alluvium surcharge is taken into account, this mine entry has a panel width-to-depth ratio of the order of 3. In this example, stability was aided by the presence of a strong sandstone stratum some 3 m thick in the immediate roof. This was not the case for the situation shown in features Fig. 3.34. Due to the large mining height, weak superincumbent strata and very shallow depth of mining, the sinkhole void in the latter case was very large and open to the mine workings, necessitating that it be backfilled. The backfilling method should be risk assessed for reasons apparent in Fig. 3.34. A more extensive layout of collapsed bords and intersections at shallow depth is shown in Fig. 3.35. Figure 3.36 shows the plug-like surface appearance of a fatal overburden failure event associated with pillar extraction at shallow depth.

The sensitivity at shallow depth of panel stability to small changes in geometry is demonstrated by considering a pillar extraction panel that has retreated 42 m without caving. The extraction of a 8 m wide pillar line (fender) from a 6 m wide roadway would result in the panel W/H ratio increasing by only 0.047 at a depth of 300 m but by 0.47 at a depth of 30 m. The impact of this change at shallow depth is illustrated by the case study shown in Fig. 3.37, in which pillar extraction at a depth of 30 m resulted in an open goaf measuring in excess of 70 m by 90 m. The production supervisor was fatality injured when, during the process of extracting a 6 m wide lift from the pillar, the entire area fell suddenly as a

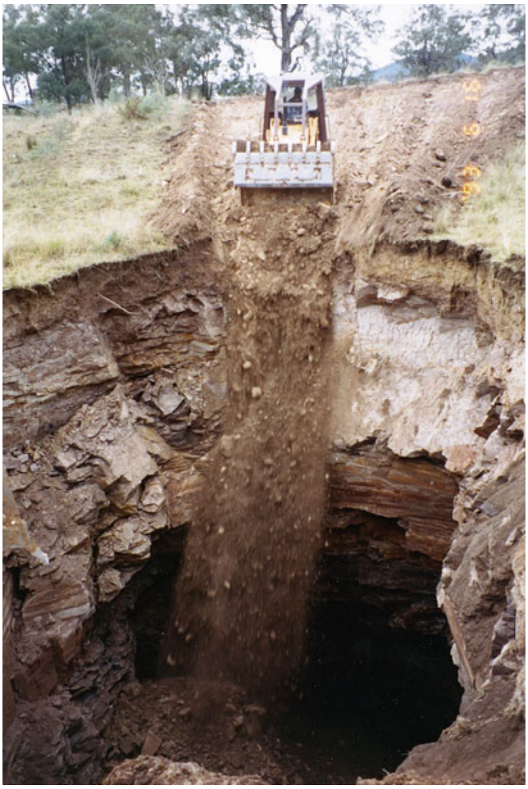


**Fig. 3.34** Intersection failure over thick seam bord and pillar workings at shallow depth

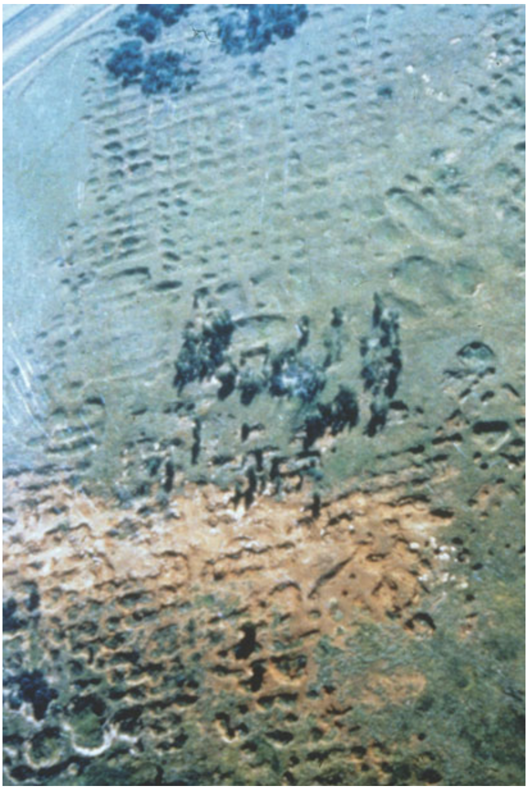


**Fig. 3.35** Surface expression of a mixture of intersection falls and bord collapses over bord and pillar workings at shallow depth

plug to the surface, generating a large windblast. The supervisor had apparently positioned himself to watch for warning signs of a goaf fall, of which there appears to have been none.

The panel had commenced against a fault plane in order to encourage the early development of caving and it is thought that heavy rainfall leading to water ingress down the fault plane at the time of the incident was a contributing factor. Water ingress along geological structures is a known trigger for the failure of both pillars and excavations in all forms of underground mining. Wagner (1991), for example, reported that a reduction in the coefficient of friction on a fault plane due to the ingress of rainwater was a contributing factor to a regional collapse of chrome mine workings in South Africa.

Butcher and Kirsten (1999) provide a detailed appraisal of managing the risk of longwall extraction at shallow depth. The following potential hazards were identified on the basis of risk assessment:

- goaf hang up followed by sudden collapse to the surface;
- geological features parallel or sub-parallel to the face, resulting in sudden and severe loading of the face supports;
- sudden ingress of water or unconsolidated material from the surface;
- ingress of surface flood water during heavy rain and storms; and
- loss of ventilation through surface cracks, leading to possible spontaneous combustion events.

Actions and controls emanating from this risk assessment approach were:

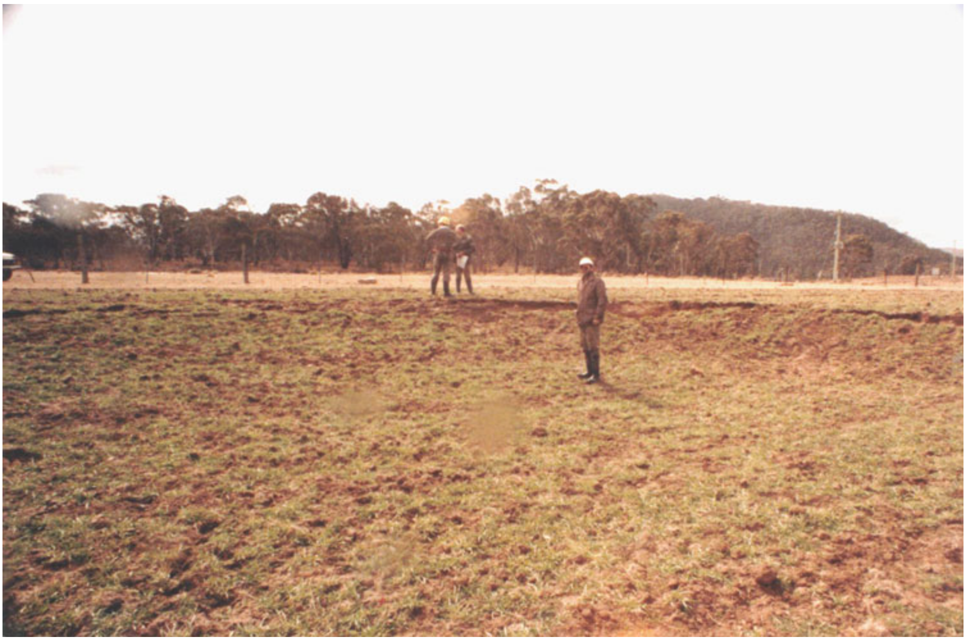
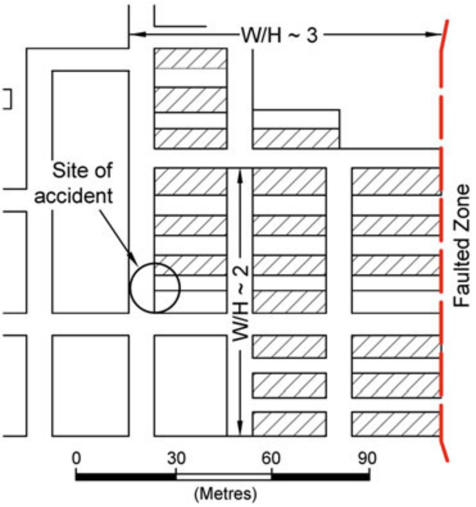


**Fig. 3.36** Surface expression of a sudden overburden failure event associated with pillar extraction panel at shallow depth



Mining height 2.4m.
Depth 20-30m.
Standing goaf > 60x90m.
Therefore minimum W/H>2.
No rib spall, roof sag, floor heave or observable deflection of props prior to fall.
Mass vertical movement to surface.
Windblast.
Conditions excellent around edge after fall.

**Fig. 3.37** Conditions associated with a fatal sudden collapse incident at shallow depth (Adapted from Galvin et al. 1994)

- thorough detailed geological mapping of gate roads and longwall face;
- extensometer monitoring of intersections and gateroads;
- in situ stress monitoring; (Aside: The effectiveness of this measure warrants careful consideration in shallow depth situations. The reliability and accuracy of the stress measuring equipment when installed in weathered rock may not be of an acceptable standard. In any case, stress and strain changes prior to the onset of instability may be low and may even fall within the sensitivity limit of the stress measuring equipment.)
- surface borehole extensometers when depth of cover was less than 35 m;
- regular monitoring and review of longwall support leg pressures and differential pressure between front and rear legs;
- monitoring face conditions each shear and implementing safety procedures immediately conditions fell outside a stated normal operating condition;
- daily surface subsidence monitoring;
- sealing of surface cracks along surface flow paths;

- additional surface inspections triggered by inclement weather to verify adequacy of surface diversions;
- emergency response and contingency plans.

These actions and controls were all encapsulated in a Shallow Mining Risk Management Plan supported by a Trigger Action Response Plan (TARP). This type of risk management approach is strongly advisable when undertaking secondary extraction at depths of less than about 50 m.

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# Pillar Systems

# 4

**Abstract**

Underground coal mining is characterised by the formation of extensive areas of pillars. The history of mining continues to be blemished by the failure of pillar systems, often with catastrophic consequences. Examples and their underpinning causes are presented throughout the chapter. Most incidents reflect that pillar system design is deceptively complex, with behaviour and stability governed by both the regional environment, which determines the load acting on the pillars, and the local environment, which determines the strength of the pillars. There is a range of pillar system design methodologies, all having limitations and none being applicable to all circumstances.

This chapter presents the structure for a functional, risk based approach to pillar design. It then builds on the fundamental physical and mechanical principles established in Chaps. 2 and 3, supported by a comprehensive review of historical and contemporary research outcomes, in presenting methodologies for estimating pillar system loads and strengths. Typical factors of safety for pillar systems are noted, with a strong emphasis placed on proceeding to probabilistic based design approaches. An example is provided of one such approach that has proved to be very successful.

Thereafter, types of coal pillar failure modes are reviewed. All of this information provides the basis for then discussing the complexity of coal pillar system behaviour and for signalling out specific critical and practical aspects of coal pillar design for more in-depth discussion. The chapter is supported by an appendix that discusses the application of civil engineering bearing capacity theory to coal pillar foundation performance.

**Keywords**

Bearing capacity failure • Bearing capacity formulae • Bieniawski formula • Bieniawski-Mark formula • Bieniawski-PSU formula • Bump • Coal burst • Confined core concept • Diamond shaped pillars • Domino failure • Dynamic failure • Effective pillar width • Floor heave • Foundation failure • Ground response curve • Highwall pillars • In situ pillar tests • Interpanel pillar • Parallelepiped pillar • Pillar failure modes •

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Pillar foundations • Pillar functions • Pillar interfaces • Pillar run • Pillar strength • Pillar system • Pillar system stiffness • Pillar types • Pillar working load • Pressure burst • Probabilistic design • Probability of failure • Rib behaviour • Risk based pillar design • Safety factor • Salamon and Munro formula • Seam specific strength • Squat pillar strength • Tributary area theory • UNSW formulae • Width-to-height ratio

## 4.1 Introduction

The mining of tabular mineral deposits is characterised by the formation of extensive areas of pillars, the design of which can be deceptively complex. Pillar behaviour and stability are governed by two interactive components, namely the regional environment which determines the load acting on the pillars (that is, the pillar working stress) and the local environment which determines the strength of the pillars. There is a range of pillar design methodologies, with all having limitations and no one methodology being applicable to all circumstances. None can be labelled ‘proven’.

A mine pillar is made up of five primary components which collectively constitute the **pillar system**. These are:

- the in-seam element, which is generally referred to as ‘the pillar’;
- the pillar/roof interface(s);
- the immediate roof strata (typically within 10 m);
- the pillar/floor interface(s), and
- the immediate floor strata (typically within 10 m).

The history of mining is and continues to be blemished by the failure of pillar systems, many with catastrophic consequences for human life. Between 1919 and 1959, seven regional pillar collapses occurred in iron ore mines in the Lorraine district of France. All were sudden and caused severe windblasts and seismic events, with 29 persons killed in two of these incidents. In recent times, abandoned workings in this region have been allowed to flood and this has triggered a new round of pillar instability with associated damage to surface infrastructure. There is also a history of pillar collapses in German potash mines, with an event in 1958 causing perceptible ground movement for an average radius of 19 km (Salamon 1974). In 1996, pillars in a potash mine in the former East Germany collapsed suddenly over an area of 2.5 km$^2$, resulting in a magnitude 4.8 seismic event (Wagner 2012).

The collapse of Coalbrook Colliery in South Africa in 1960 constitutes one of the most catastrophic incidents. Some 4400 pillars failed in a 5 min period and around 7000 in total in a 20 min period, resulting in 437 deaths. In the late 1980s, a pillar collapse in a South African chrome mine resulted in a fatality on the surface when a person was hit by a locomotive ejected from the mine by the associated windblast (Wagner 1999). Malan and Napier (2011) report on a number of other pillar failures in this mining sector. Six collapse events in hard rock and salt mines in the USA have been documented by Zipf and Mark (1996).

In the 10 years to 1998, 12 large pillar collapse events occurred in coal mines in the USA, each resulting in major airblast damage to mine infrastructure (Chase et al. 1994). In August 2007, pillar extraction operations at Crandall Canyon Mine in the USA initiated a regional pillar failure over a distance of some 800 m, resulting in a magnitude 3.9 seismic event. A pressure burst occurred the following day in a recovery roadway from which coal ejected in the pressure burst had been removed in an attempt to reach the working face where six mine workers were unaccounted for. Nine days later, during re-mining of a recovery roadway, another pressure burst resulted in three more fatalities (Gates et al. 2008).

A number of collapses of bord and pillar workings have been associated with floor failure. Twenty-six miners had to be rescued through a surface to seam borehole in Swaziland in 1991 after they were trapped by such an event. A pillar collapse over an area of 1.6 $km^2$ in a trona mine in the USA in 1995 generated a magnitude 5.3 seismic event, resulting in one fatality. Latilla (2003) attributed this collapse to failure of the floor foundations of the pillars.

Many serious incidents of coal pillar system instability have occurred in Australia over more than 2 centuries. The Hamilton pit disaster in Newcastle in 1889 resulted in 11 fatalities and a landmark recommendation to increase pillar size. In 1908, a Royal Commission was held into three pillar instability events that caused severe structural damage to Newcastle Cathedral and surrounding buildings.

Between 1980 and 2014, 33 unplanned pillar system failures are known to have occurred in bord and pillar and partial pillar extraction workings in NSW and Queensland coal mines, including five in workings formed after 2002. Fortuitously, none resulted in fatalities but financial losses and surface damage were severe in some instances. At least six of the events occurred suddenly in active working sections with minimal or no warning and were due to failure of the in-seam coal element. On four occasions, the mines were idle and on two occasions workers had been assigned to other production districts just prior to the incidents.

Another ten events were associated with failure of the roof or floor foundations of pillar systems. Two of these developed backbye and ran into active mining areas with little or no warning, wedging mobile equipment between the roof and the floor. Three others incidents resulted in the loss of main development roadways in active mines, including the loss of access to life-of-mine reserves for two longwall units in one mine. The most recent incident was associated with a partial pillar extraction design that relied on the pillar system supporting the weight of a strong and massive roof strata that bridged over extended mining spans. The instability was detected some six years after implementation of the design and two years after the mine was closed and allowed to flood.

Although mechanised longwall mining now accounts for a high proportion of underground coal production, safety and productivity are still contingent on the behaviour of coal pillar systems. This behaviour can affect many factors, including the stability of main development panels, chain pillar design, gateroad support, local and regional mine stability, and surface subsidence control.

Most safety and production difficulties related to instability of the pillar system can usually be attributed to either:

- a design focus restricted to the strength of the in-seam element (pillar);
- the inappropriate application of a pillar system design procedure to conditions outside its intended purpose or operational range; and
- non-compliance with the designed mine layout.

A knowledge of pillar system behaviour provides a basis for understanding how pillars influence the local and regional stability of underground mine workings, for appreciating the uncertainties associated with pillar design, and for managing the associated risks accordingly.

## 4.2 Functional, Risk Based Approach to Pillar Design

Pillars perform four basic functions in underground coal mining. They provide:

- natural temporary or permanent support to the surrounding strata;
- a buffer zone between adjacent excavations to control interaction between their respective stress fields;
- a physical barrier to restrict fluid flow between excavations; and
- a control for managing the magnitude and extent of surface subsidence.

These functions can be associated with a range of mining methods and usually overlap to some extent, giving rise to a variety of pillar types as shown in Table 4.1. Protective and barrier pillars fall at one extreme, with their design based primarily on the lateral distance, or pillar width, required to dissipate the effects of mining. This often results in the pillars having such a large width-to-height ratio that they are also permanently stable and provide regional support. Fenders and stooks fall

**Table 4.1** Types, functions and typical life expectancy of coal pillars

| Type | Range of functions | Typical life expectancy |
|---|---|---|
| Protective | Provide a zone of protection against ground movement near sub-surface and surface infrastructure and natural features | From life-of-infrastructure to permanent |
| Barrier | Provide a zone of separation of sufficient width between two sets of workings to limit interaction between their respective stress fields | From life-of-mine (10–40 years) to permanent |
| | Provide a solid barrier against inrush, gas migration and spontaneous combustion | |
| | Protect sub-surface and surface natural and man-made infrastructure from mining-induced ground movement | |
| Main Development | Local or regional load bearing structure | – Life-of-mine (10–40 years), or<br>– From life-of-infrastructure to 'permanent' |
| | Restrict strata displacement around main development roadways to safe and serviceable levels | |
| | Act as ventilation stoppings | |
| | Protect sub-surface and surface natural and man-made infrastructure from mining-induced ground movement | |
| Panel | Local load bearing structure | – Life-of-panel (1–2 years), or<br>– From life-of-infrastructure to permanent |
| | Provide roof, rib and floor stability within a panel for duration of production | |
| | Restrict sub-surface and surface ground movement | |
| Interpanel | Regional load bearing structure between adjacent panels | – Life-of-mine (10–40 years), or<br>– From life-of-infrastructure to permanent |
| | Provide a sufficiently wide separation between two adjacent panels to limit the interaction of their respective stress fields | |
| | Restrict the spread of a pillar system instability | |
| | Provide a solid barrier against inrush, gas migration and spontaneous combustion | |
| Chain | Protect companion gateroads from abutment stress | – 1–3 years, or<br>– Life-of-infrastructure, up to permanent for partial extraction systems |
| | Provide a ventilation pathway and 2nd egress | |
| | Function as a goaf seal | |
| | Sometimes used to provide regional support and restrict sub-surface and surface ground movement | |
| Yield | Localised, low stiffness support | – 1–3 years |
| | Limit damage to immediate roof and floor strata, mitigate pressure bursts (coal bumps, rock bursts) | |
| | Provide localised stress relief around a roadway | |
| | Improve percentage extraction in some bord and pillar mining layouts | |
| Highwall | Temporary, local support to current drivage or punch | – Hours to days<br>– Sometimes permanent |
| | Sometimes used to provide regional support | |
| Fender | Local support and goaf edge control | 3–5 days, then encouraged to fail |
| | Break off point for cantilevering roof | |
| | Barrier against a goaf fall. | |
| Stook | Local support to protect retreat path from or through an intersection | 1–3 days, then encouraged to fail |
| | Goaf edge control | |

at the other extreme, being remnant pillars that are only intended to provide temporary protection from roof falls during extraction of the mother pillar. Their width-to-height ratio is deliberately minimised to encourage their failure very soon after they have fulfilled this function in order that caving is not impeded.

Tolerable risk of failure is an important consideration when designing pillars that are required to fulfil a support function. Historically, design has been based on the concept of safety factor, introduced in Sect. 2.7.4. In order to quantify risk associated with design, safety factors need to be correlated to the probability of a successful

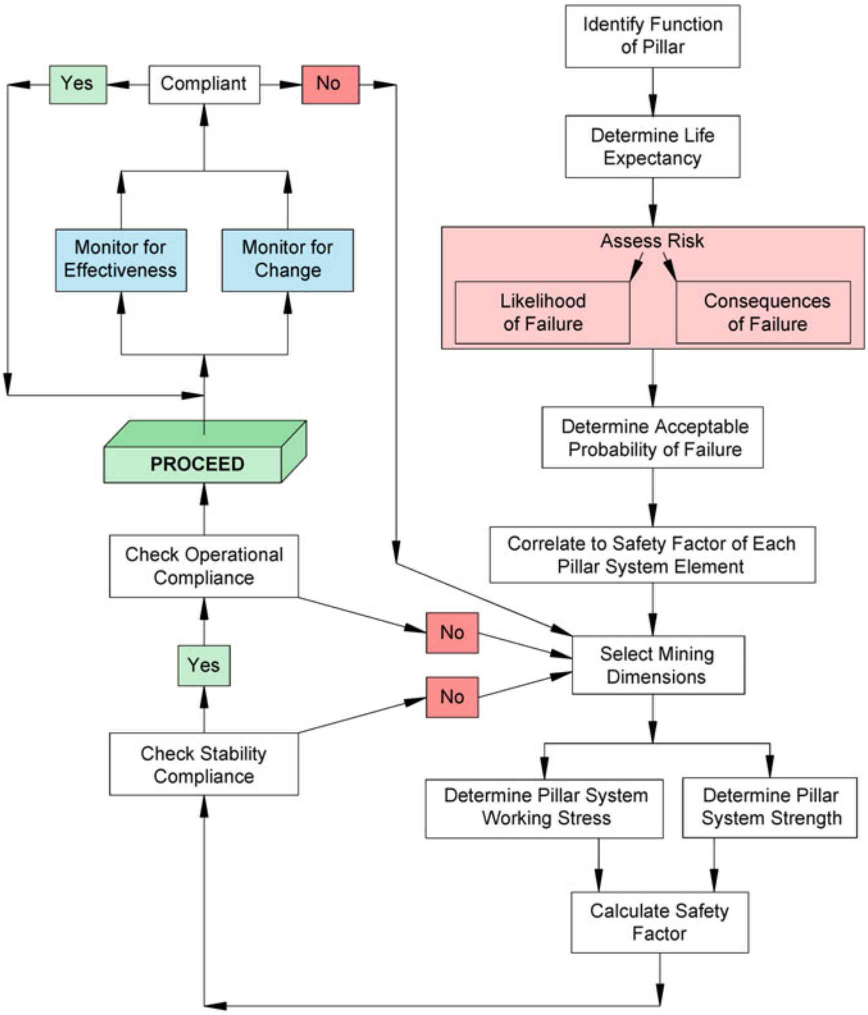

Identify Function of Pillar
Determine Life Expectancy
Assess Risk
Likelihood of Failure
Consequences of Failure
Determine Acceptable Probability of Failure
Correlate to Safety Factor of Each Pillar System Element
Select Mining Dimensions
Determine Pillar System Working Stress
Determine Pillar System Strength
Calculate Safety Factor
Check Stability Compliance
No
Yes
No
Check Operational Compliance
PROCEED
Monitor for Effectiveness
Monitor for Change
Compliant
Yes
No

**Fig. 4.1** The steps associated with a risk-based, functional approach to pillar design

outcome, otherwise subjective judgements of risk have to be made.

Figure 4.1 summarises the steps involved in applying a quantitative risk-based, functional approach to pillar system design.

## 4.3 Pillar Working Stress

### 4.3.1 Pillar System Stiffness

Equation 2.3 provides the basis for establishing that the stiffness of a coal pillar increases with an increase in its modulus and area and decreases with an increase in its mining height. Similarly, the stiffness of the superincumbent strata increases with an increase in its modulus and thickness and decreases with an increase in its span, although not in such a simple manner as defined by Eq. 2.3. Effectively, therefore, stiffness encapsulates geology, depth of mining, panel width, pillar width and mining height. Hence, these factors are variables in determining stress magnitudes and distributions in and around mine workings.

In Sect. 2.8.2, it was noted that when a beam has more supports than necessary to achieve a state of equilibrium, the load reactions cannot be determined by simply balancing the forces and moments and, therefore, the situation is said to be ‘statically indeterminate’. Resolution of load and displacement distributions and reactions in these circumstances requires knowledge of the complete stiffness properties of both the beam and the supports. Nearly all coal pillar loading systems fall into this category, with individual pillar load being a function of the stiffness, or load-deformation characteristics, of both the pillar and the surrounding strata.

The difficulty in estimating loads and displacements in these situations can be illustrated by visualising the superincumbent strata as an elastic beam resting on a coal seam. When an excavation of height, h, is formed, the beam deflects into it by an amount, $\delta_b$, shown in Fig. 4.2a. Beam deflection increases as:

- span increases, equating to an increase in the width, W, of an excavation;
- thickness decreases, equating to a decrease in the depth of mining, H; and
- elastic modulus decreases, equating to a decrease in the effective modulus, $E_o$, of the superincumbent strata.

Beam deflection is reduced when a pillar is left in situ at mid-span (Fig. 4.2b). This pillar behaves as a spring and compresses under the weight of the beam by an amount, Δh, which must equal the beam deflection, $\delta_b{}'$ (Fig. 4.2c). The more the beam deflects, the greater the opposing force (or load) generated in the spring. The manner in which the stiffness of both the superincumbent strata and the coal pillars interact to determine pillar load can be visualised by replacing the coal pillars in a mining layout with springs of corresponding stiffnesses, as illustrated in Fig. 4.3. This analogy forms the basis of some numerical modelling techniques. It illustrates how the stiffer pillars attract load and shield the smaller adjacent pillars from load.

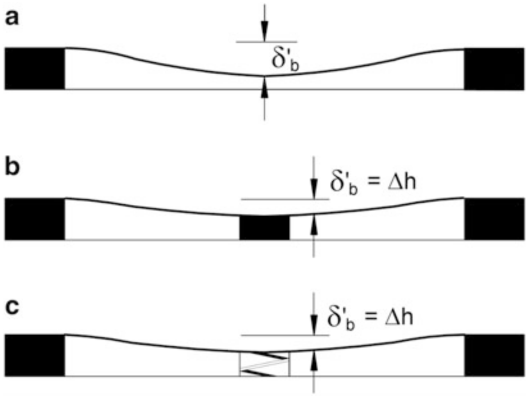


**Fig. 4.2** Elastic beam – pillar interaction model. (**a**) No pillar support to beam, (**b**) Beam supported by a pillar at mid-span, (**c**) Supporting pillar replaced by a spring of equivalent stiffness

In the most general case of a parallelepiped shaped pillar of cross-sectional area, $A_p$, and height, h, shown in Fig. 4.4, the load generated in the pillar by the deflection of the roof beam is given by Eq. 4.1.

$$\begin{aligned} \textit{Pillar Load} &= L_p = \frac{E_c A_p \Delta h}{h} \\ &= \frac{E_c (w_1 \sin\theta\, w_2)\Delta h}{h} \\ &= \frac{E_c (w_1 w_2 \sin\theta)\Delta h}{h} \end{aligned} \tag{4.1}$$

where

$E_c$ = *elastic modulus of coal*
$A_p$ = *cross-sectional area of a parallelepiped shaped pillar*
$\Delta h$ = *compression of the pillar*
$h$ = *pillar height*
$w_1$ = *length of shortest pillar side*
$w_2$ = *length of longest pillar side*
$\theta$ = *the smaller internal angle between pillar sides* ($\theta \leq 90°$)

Equation 4.1 shows how the prediction of pillar load requires knowledge of the convergence (or elastic roof to floor closure) distribution at seam level. With few exceptions, numerical modelling is required to simulate the stiffnesses

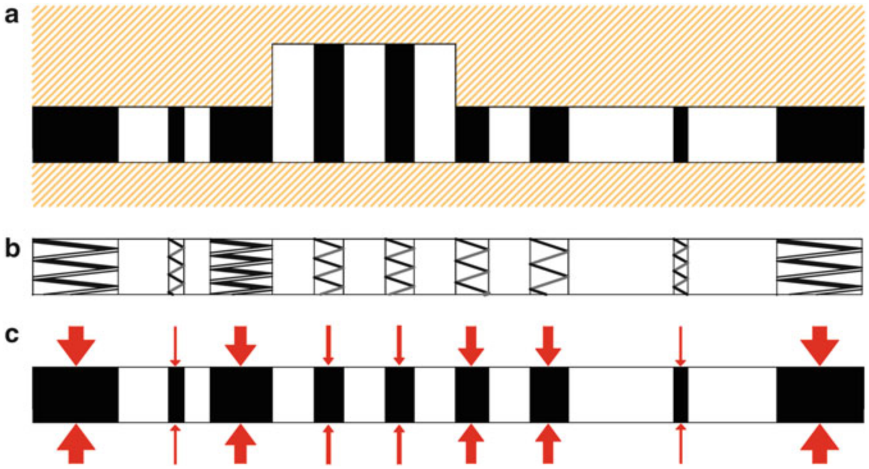
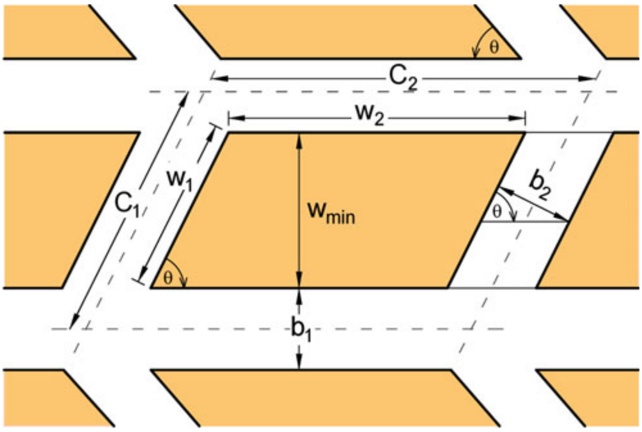


**Fig. 4.3** Visualisation of load sharing in a pillar system utilising a beam and spring model. (**a**) Variation in pillar area and height, (**b**) Equivalent spring stiffness, (**c**) Load distribution

**Fig. 4.4** Geometry and dimensions associated with a uniform layout of parallelepiped pillars



of the pillars and the stiffness of the overburden in order to determine this convergence distribution.

### 4.3.2 Regular Bord and Pillar Layouts

Fortunately, a regular bord and pillar layout, such as that shown in Fig. 4.5, constitutes one of the few underground coal mining layouts that approximates to a statically determinant situation provided that:

- the width of a panel of pillars, $W_p$, is sufficiently large that the stiffness of the superincumbent strata is reduced to zero to result in deadweight loading;
- the pillars are all of similar size and shape;
- the bords are all of similar width and height; and
- the mine layout is uniform.

If these criteria are satisfied, the pillars will all have the same stiffness and, therefore, can be assumed to carry an equal share of the deadweight load of the overburden within their area of influence, defined by the loci of mid-points to the surrounding pillars. This concept is referred to as **tributary area theory**, with pillar load being referred to as **tributary area load** (TAL).

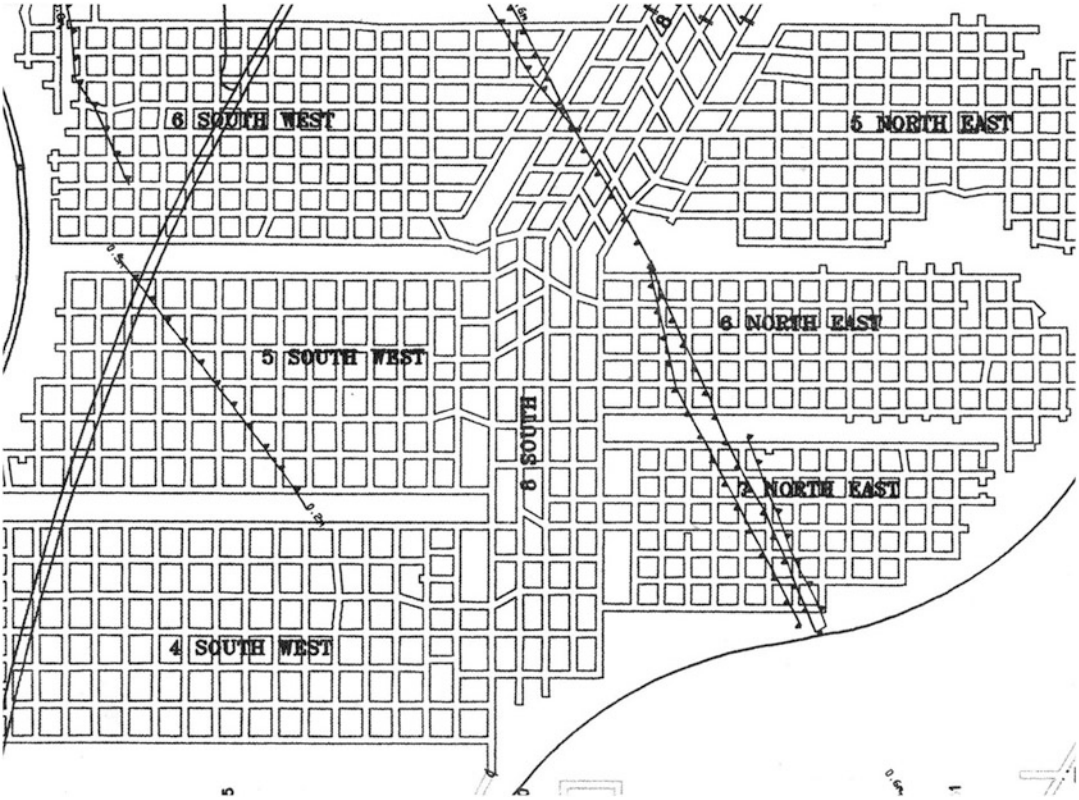


**Fig. 4.5** An example of a regular layout of bord and pillar panels, partitioned with interpanel pillars

The concept is illustrated in Fig. 4.6, which shows a pillar carrying the entire weight of the column of overburden acting within its area of influence, $A_c$.

If horizontal stratification is assumed in virgin conditions then, prior to mining, the primitive vertical load acting over the tributary area, $A_c$, of a future coal pillar is given by Eq. 4.2. It should be noted that as a matter of convenience and conservatism, the value of depth, H, in some popular pillar design formulations is measured to the floor of the mine workings (for example, those of Salamon and Munro (1967) and Salamon et al. (1996)).

$$\begin{aligned} Primitive\ vertical\ load &= primitive\ vertical\\ &\quad stress\ x\ area\\ &= \rho_o g H A_c\\ &= \rho_o g H\ C_1 Sin\theta\ C_2 \end{aligned} \tag{4.2}$$

where

$\rho_o = overburden\ density\ (kg/m^3)$
$g = gravitational\ acceleration\ (m/s^2)$
$H = depth\ to\ base\ of\ seam\ (m)$

Pillar formation results in the same load now having to be carried by the pillar and, therefore, acting over the cross-sectional area of the pillar, $A_p$. Hence, **average vertical pillar stress** is given by the following formulations:

$$\begin{aligned} Average\ pillar\ stress,\ \sigma_{aps} &= \frac{pillar\ load}{pillar\ area}\\ &= \frac{\rho_o g H\ x\ A_c}{A_p}\\ &= \frac{\rho_o g H A_c}{A_p}\\ &= \frac{\rho_o g H\ C_1 Sin\theta\ C_2}{w_1 Sin\theta\ w_2} \end{aligned} \tag{4.3}$$

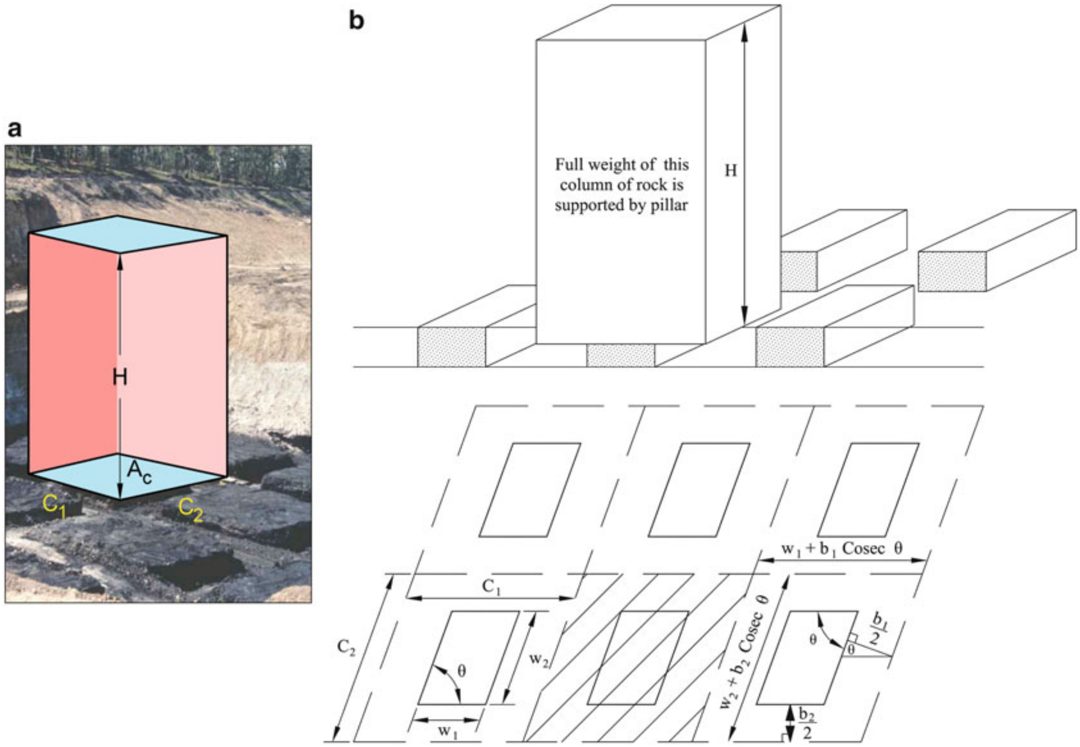


**Fig. 4.6** The concept of tributary area theory. (**a**) Square pillars, (**b**) Parallelepiped pillars

$$= \frac{\rho_o g H C_1 C_2}{w_1 w_2} \tag{4.4}$$

$$= \frac{\rho_o g H}{1 - e} \tag{4.5}$$

where

$$e = areal\ extraction\ (as\ a\ fraction)$$

$$= \frac{(C_1 Sin\theta)C_2 - (w_1 Sin\theta)w_2}{(C_1 Sin\theta)C_2}$$

$$= 1 - \frac{w_1 w_2}{C_1 C_2} \tag{4.6}$$

For most practical coal mining applications, these equations can be simplified by approximating gravitational acceleration to 10 m/s$^2$ and the density of the overburden to 2500 kg/m$^3$ (or 0.025 MPa). This corresponds to the primitive vertical stress, $\sigma_{vp}$, increasing by 1 MPa, or 100 t/m$^2$, for each 40 m increase in depth. Hence:

$$Primitive\ vertical\ stress,\ \sigma_{vp} \cong 0.025H\ (MPa) \tag{4.7}$$

$$Average\ pillar\ stress,\ \sigma_{aps}$$

$$\cong \frac{0.025HC_1C_2}{w_1 w_2}\ (MPa) \tag{4.8}$$

$$\cong \frac{0.025H}{1 - e}\ (MPa) \tag{4.9}$$

Because tributary area theory is premised on the stiffness of the overburden being zero, it can overestimate the load on all pillars in a panel that is not very wide relative to depth. Irrespective of panel width-to-depth ratio, in most situations tributary area theory overestimates loads on pillars towards the perimeter of a panel because of the effects of panel abutments on displacement of the superincumbent strata close to the edges of an extraction panel. These aspects are illustrated in Fig. 4.7 for a roadway/pillar width geometry that

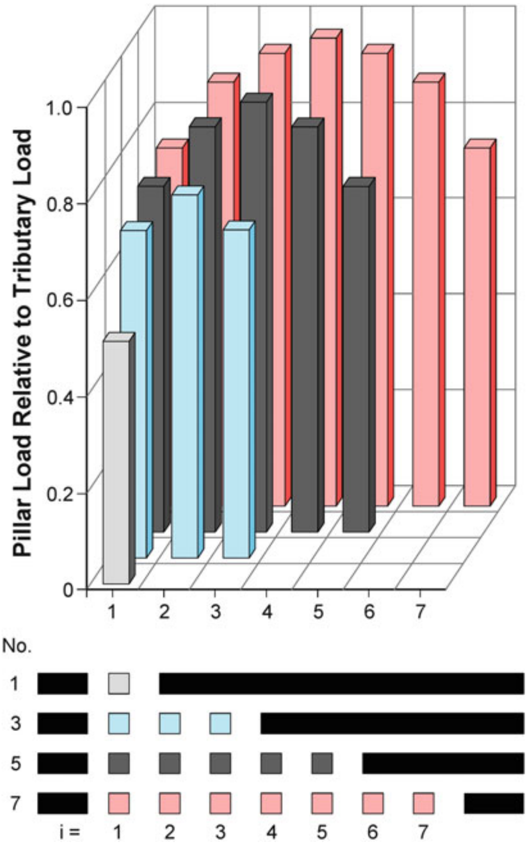


**Fig. 4.7** An example of the influence of panel width on pillar load (after Salamon 1992b)

achieves an areal extraction of 75 %. The plots show the variation in numerically calculated pillar load, expressed as a proportion of tributary area load, as panel width is increased. The pillar loads range from 50 % of tributary area load when the panel is one pillar and two bords wide, to 95 % for a panel that is seven pillars and eight bords wide.

When invoking tributary area theory, it is important to appreciate that:

- The technique only produces the average pillar working stress, which is assumed to be constant across the pillar. In practice, stress magnitude varies across a pillar, with some portions experiencing less than the average pillar stress and other portions experiencing more.
- The loads acting on pillars in the centre of a panel of pillars generally approach full deadweight loading once overall pillar panel width, $W_p$, exceeds 1–1.5 times depth. However, larger panel spans may be required to achieve full deadweight loading when the overburden contains massive, stiff strata.
- Unless the properties of the superincumbent strata have been determined in detail and are known to be consistent across the mining layout, it is an advisable risk management measure to base the design of panel pillars on full tributary area load irrespective of the nature of the surrounding strata and the overall pillar panel width-to-depth ratio.
- Pillar working stress increases exponentially with percentage extraction, with incremental change for a given increase in extraction becoming greater with depth, Fig. 4.8. For example, a ten per cent increase in extraction from 30 to 40 % results in a 16.7 % increase in average pillar stress while the same 10 % increase in extraction from 70 to 80 % results in a 50 % increase in pillar stress. It should be noted that, contrary to what Fig. 4.8 might suggest, problems arising from over-extraction are most common and serious in shallow mine workings. This is because, firstly, pillars tend to be small at shallow depth and, therefore, the impact on pillar stability of a given amount of over-extraction is greater; and, secondly, high percentage extraction bord and pillar layouts are not feasible at depth as the coal pillars cannot support the very high overburden loads. Vigilant management of mining dimensions is particularly important at shallow depth.

### 4.3.3 Irregular Bord and Pillar Layouts

Pillars of irregular size, shape and height, such as those shown in Fig. 4.9, were a common feature of bord and pillar mining in the days of hand mining and drill and blast mining operations. A need to assess pillar load in irregular mine layouts often arises when considering the potential for surface subsidence over old workings or

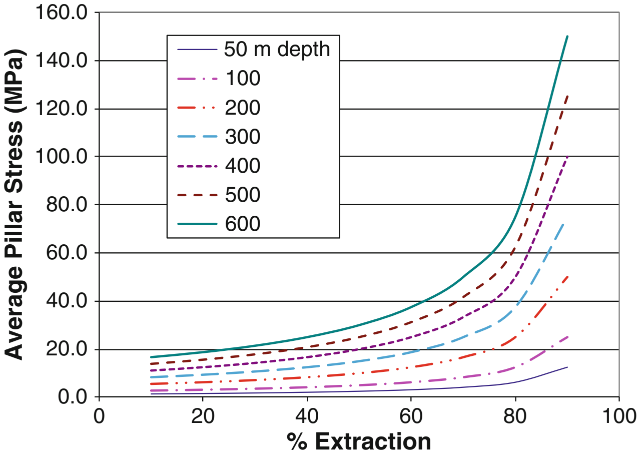


**Fig. 4.8** Plots showing how average pillar stress increases exponentially with increasing percentage extraction and depth

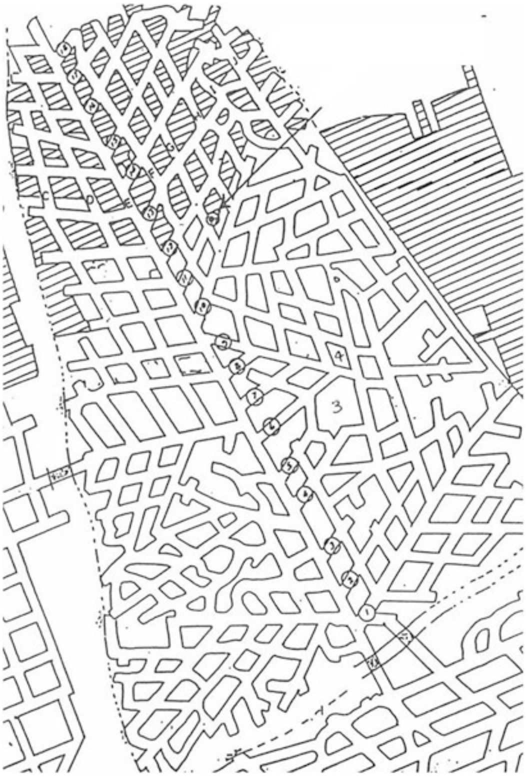


**Fig. 4.9** An example of irregular bord and pillar workings (After Anderson 1993)

interaction between these workings and workings in adjacent seams.

Two pitfalls often associated with attempting to use analytical or empirical techniques to assess the stability of irregular bord and pillar workings are a focus on the pillars with the smallest plan area and the application of tributary area theory. While visually, the small pillars might appear to be the weakest links in the system, overall panel stability may in fact be controlled by the pillars with the larger plan areas and/or the lower mining heights because they are stiffer and will generate a higher load in response to convergence of the superincumbent strata. The smaller, less stiff pillars may be protected from load until the stiffer pillars fail, at which point failure of the mine layout is assured.

It is advisable to utilise three-dimensional numerical modelling techniques when assessing the stability of irregular bord and pillar layouts. Even then, care is required since outcomes can be very sensitive to input parameters, especially the moduli of the superincumbent strata and the coal. Outcomes are valuable in providing insight into the relative load sharing between pillars but absolute load values should be treated with caution. Parametric and sensitivity analysis are recommended.

If an analytical or empirical stability assessment approach is attempted, consideration should be given to basing it on a mining layout that has had the small pillars removed. A point which cannot be over-emphasised is that safety factor is not a valid criteria for assessing load transfer between pillars.

## 4.4 Pillar System Strength

### 4.4.1 Defining Pillar Strength and Failure

**Strength** was defined in Sect. 2.5.4 as the maximum or peak stress that a structure can sustain, or its maximum resistance to deformation. Because stress magnitude varies across the width of a coal pillar, pillar strength is usually expressed in terms of **peak average stress**, calculated by dividing the peak working load by the plan area of the pillar.

Figure 4.10 shows the manner in which pillar behaviour is influenced by pillar width-to-height ratio. After its maximum resistance to deformation has been exceeded, a pillar with a low width-to-height ratio undergoes strain softening and progressively and permanently unloads with ongoing displacement as depicted by curve (a). Usually, once pillars of this geometry start to unload they are no longer able to perform their intended function and, therefore, are generally considered to have 'failed'. As pillar width-to-height ratio is increased in the interval between curve (a) and curve (c), a pillar still sheds load when its strength is exceeded but then it unloads at an increasingly slower rate with increase in width-to-height ratio. Curve (c) corresponds to that pillar width-to-height ratio that results in the pillar attaining a state of near constant load carrying capacity, or plastic yield. Curve (d) shows how with further increases in width-to-height ratio, pillar resistance to deformation may still initially peak and drop, resulting in load shedding, but then the pillar goes on to exhibit strain hardening characteristics and to accept load indefinitely. If the pillar width-to-height ratio is sufficiently large, the pillar effectively behaves as an abutment and, as shown by curve (e), strain hardening may develop without the pillar shedding load.

At width-to-height ratios of less than about four, pillar failure is unmistakeable. It is usually associated with pillar crushing and an obvious reduction in seam height, which are frequently accompanied by caving of the superincumbent strata. Surface subsidence is readily measurable, if not obvious to the naked eye. Depending on the stiffness of the overburden, pillar panels may totally disintegrate. However, as pillar width-to-height ratio increases, it becomes increasingly difficult to nominate a particular value of peak pillar stress and to define the point of structural failure. In practice, this point is frequently determined, mistakenly so, on the basis of the appearance of the pillar sides and the serviceability of the excavations surrounding the pillar. Based on experience supported by field instrumentation and numerical modelling, it appears that pillars with a width-to-height ratio greater than six can be in a structurally failed state while still remaining functional, and, conversely, they can be in an unserviceable state while still being structurally stable.

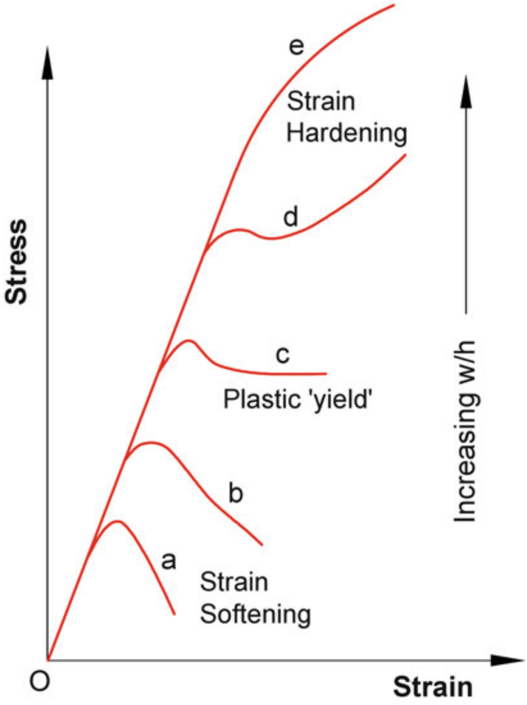


**Fig. 4.10** Effect of width-to-height ratio on the stress-strain characteristics of a coal pillar

Brady and Brown (2006) approached this conundrum by noting that the onset of fracturing is not necessarily synonymous with failure or with the attainment of peak strength. They suggested that an alternative engineering approach is to say that the rock has failed when it can no longer adequately support the forces applied to it or otherwise fulfil its engineering function. This may involve considerations of factors other than peak strength and that, in some cases, excessive deformation may be a more appropriate criterion of failure.

This text subscribes to the alternative engineering approach suggested by Brady and Brown (2006). However, it cannot be applied clinically when discussing coal pillar systems because of the legacy associated with the variable and ill-defined meanings assigned in past studies and research into pillar strength to terms such as ‘failed’, ‘collapsed’, ‘stable’, ‘unfailed’, ‘serviceable’ and ‘functional’. The manner in which the terms **pillar failure** and **pillar collapse** are often used loosely and interchangeably adds to the confusion. In this text, the term ‘pillar collapse’ refers to failure situations where the pillar has, for all practical purposes, lost its structural integrity and been consumed by a collapse of the mine workings.

Hence, the reader needs to be perceptive to terms used in literature to describe states of pillar stability and instability and exercise intellectual discretion in interpreting their intended meaning. Fortunately, because past research and incidents have been weighted towards pillars of small width-to-height ratio, the use of the term ‘pillar failure’ in these cases is often synonymous with both the pillars exceeding their point of maximum resistance to deformation and with them no longer being capable of performing their intended engineering function. Furthermore, the majority of documented pillar failure events of the past constitute pillar collapses. However, this is not to say that the peak load carrying capacity of pillars with a moderate to high width-to-height ratio has not been exceeded on occasions.

Numerous investigations dating back over a century into the strength of pillar systems have produced a diversity of outcomes. These reflect investigations focused on selective components of a pillar system and on selective failure modes; the different interests of the researchers in the particular function performed by pillars; and the range of investigative techniques utilised in the studies. It is important, therefore, that the end-user of a pillar design procedure has an appreciation of the conditions and limitations associated with its derivation and application.

### 4.4.2 Geological Factors

#### 4.4.2.1 Pillar Roof and Floor Interfaces

When pillars are formed in a horizontal or near horizontal seam, the vertical stress that previously acted through the surrounding excavated rock mass is redirected through the pillars, resulting in a non-symmetrical stress distribution on the roof and floor interfaces, or end contacts, of the pillars. This stress can be resolved into vertical and horizontal components, as shown in Fig. 4.11. The average of the vertical components of overburden stress, which constitutes the average pillar stress, acts as a normal force on an interface, thus increasing its shear strength. If an interface between a pillar end and the surrounding rock mass is rough and/or well bonded, the horizontal stress component can act across the interface and provide resistance to lateral movement of the pillar. However, if a soft or weak band is present at or in the vicinity of an interface, the vertical stresses will promote lateral squeezing of the band, thereby facilitating lateral straining of the pillar and preventing build up of confinement in the central part of the pillar. Hence, the pillar is effectively weakened.

Shear forces are also generated on the interfaces. This is because the formation of an excavation removes confinement to the pillar sides and increases the overburden stress acting on the pillar, thus causing lateral elastic straining of the pillar sides due to the Poisson’s effect and, with the onset of yield, lateral inelastic dilation of the pillar. The shear strengths of the roof and floor interface generates resistance to pillar straining and dilation at these horizons

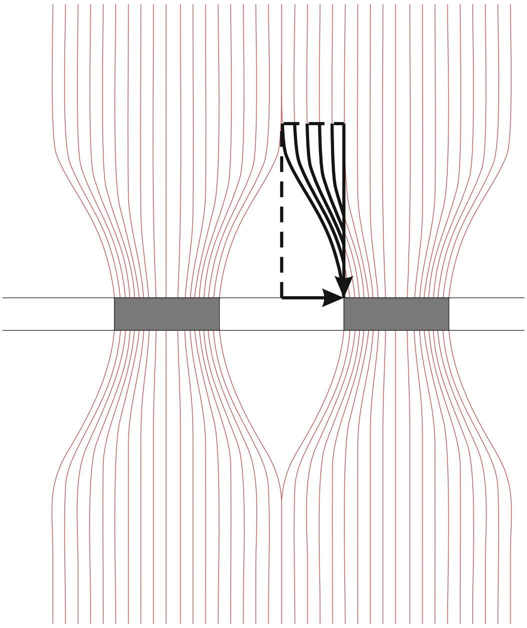
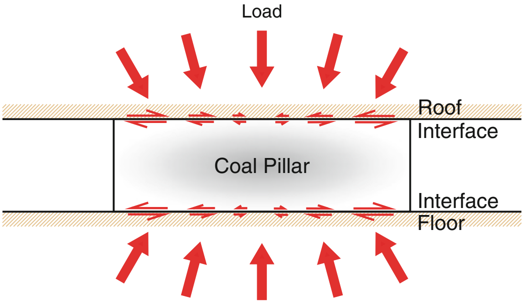


**Fig. 4.11** Components of overburden stress acting on the ends of a pillar



Under load coal pillar wants to expand laterally. Friction and cohesion on roof interfaces generate resistance to this movement.

**Fig. 4.12** Diagram showing how shear resistance to lateral pillar dilation is generated on the contact surfaces of a pillar

(Fig. 4.12). Since rock strength is a function of confinement, this phenomenon has the potential to significantly influence pillar strength, with the extent of the influence being a function of:

- depth (H), since this governs the magnitude of the normal stress acting on the interfaces;
- the bridging span (b or W) of adjacent excavations, since this influences the magnitude of the horizontal components of vertical stress induced at the pillar ends;
- pillar width and length ($w_1\text{Sin}\theta$ and $w_2$), as these parameters determine the contact area over which confining shear stress can be developed; and
- the cohesive and frictional properties of the roof and floor interfaces (which may actually comprise a series of interfaces in the general vicinity of the pillar ends).

Although the preceding engineering logic suggests that pillar strength will be influenced by the depth of mining and by excavation width, these parameters do not feature in empirical pillar strength formulae. Salamon et al. (1996) acknowledged the possible influence of excavation width, b or W, but did not evaluate this in their derivation of the UNSW pillar strength formulations. This is an example of the complex and incomplete knowledge base concerning pillar behaviour and of the opportunity afforded by numerical modelling to advance this understanding.

The influence on pillar strength of the lateral confinement generated at the pillar ends decreases with distance from the end interfaces. This is evidenced by the hourglass shape of stressed pillars in homogenous rock mass conditions, reflected to a degree in Fig. 4.13. It is one reason why the strength of a pillar of a given width decreases with increase in pillar height.

Figure 4.14 illustrates how pillar strength and behaviour can be severely modified by the presence of low shear strength interfaces at and near roof or floor contacts. These interfaces may comprise bedding planes or one or more bands of soft or weak strata. They provide limited resistance to pillar dilation, which can develop readily on natural cleat and joint planes within the coal. Interfaces can also have an adverse impact on pillar performance when located within the main body of the pillar, as illustrated in Figs. 4.15 and 4.16.

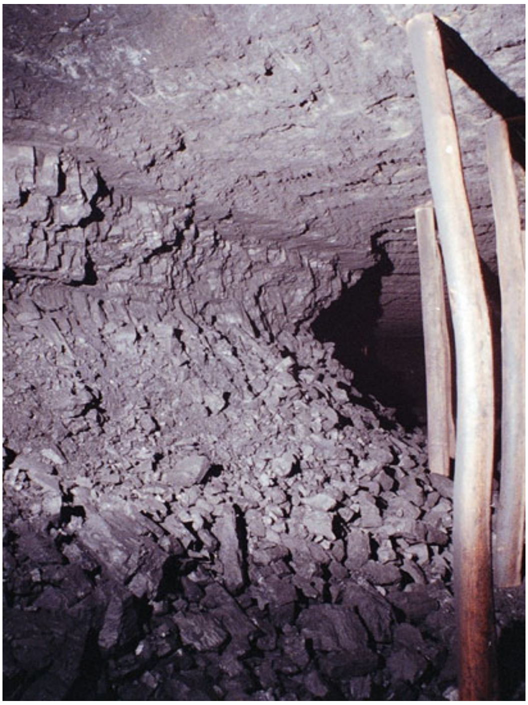


**Fig. 4.13** A highly stressed pillar in high friction and cohesion contact conditions, resulting in the pillar taking on an hourglass shape

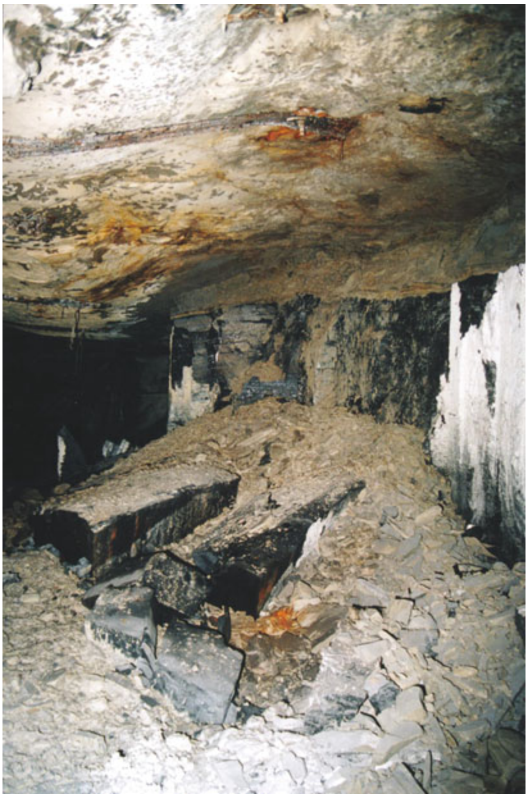


**Fig. 4.14** Blocky rib spall along cleat planes due to induced lateral tension caused by extrusion at the pillar/roof interface of a claystone with low cohesive and frictional properties

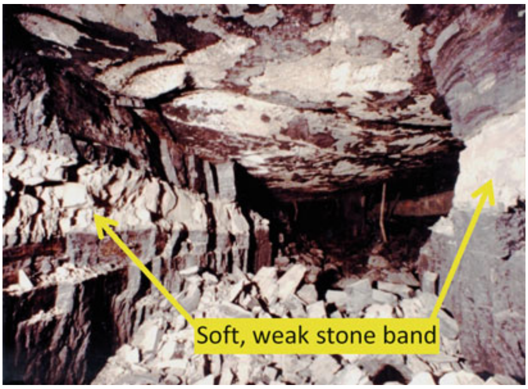


**Fig. 4.15** Rib spall caused by the extrusion of a soft and weak stone band within the extraction horizon

### 4.4.2.2 Foundations

The term **foundation** has a confused meaning in geotechnical engineering. It is used to refer to both the natural material on which the base, or **footing**, of a structure is founded, and to the footing itself. In this text, a coal pillar is considered to constitute a footing, with the strata immediately above and below the pillar constituting foundations. Pillar system performance is dependent on the foundations having the capacity to bear the load directed through the pillar.

Figure 4.17 shows an example of general bearing capacity failure of a coal pillar foundation. If the thickness of a weak foundation layer is limited or the layer is significantly weaker than adjacent low strength layers, slip surfaces may not develop throughout the full thickness of the foundation. Rather, the layer may shear on bedding planes and extrude laterally from under or over the pillar footing (Fig. 4.18). This subjects the pillar to lateral tension which, because the tensile strength of a coal pillar is minimal (and effectively zero in the presence of cleating), induces open cracks that may extend from roof to floor and through the full width of the pillar

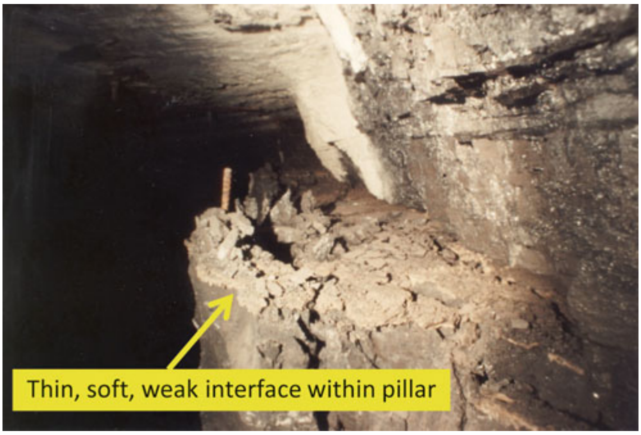


**Fig. 4.16** Pillar dilation due to sliding on a low cohesion and friction interface within the extraction horizon

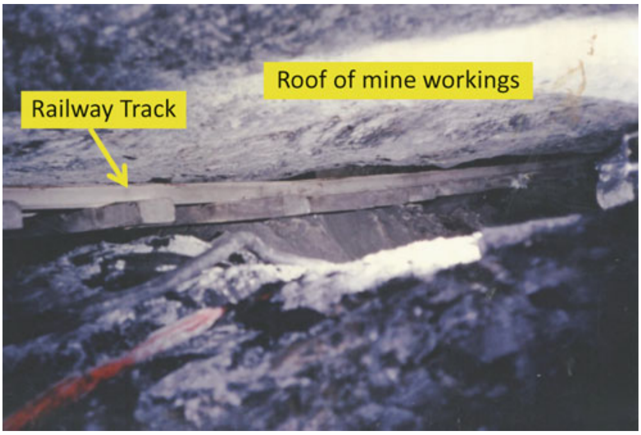


**Fig. 4.17** Bearing capacity failure of the floor in 4.5 m high bord and pillar workings, resulting in floor heave pushing the railway track hard up against the roof

(Fig. 4.19). The pillar failure mode changes from one of compressive failure under axial compressive stress to one of tensile failure under lateral extension strain, resulting in a substantial reduction in the load bearing capacity of the pillar. Some of these aspects and their consequences for pillar strength are discussed in more detail in Sect. 4.4.5.

## 4.4.3 Geometric Factors

### 4.4.3.1 Pillar Shape

In situ pillar strength tests conducted by Wagner (1974) provide insight into how pillar shape influences pillar strength. Coal pillars were loaded to destruction using a displacement controlled loading system comprising twenty five 500 tonne hydraulic jacks (Fig. 4.20). The vertical stress in each jack was recorded at different stages of compression to produce plots of the distribution of vertical stress in the centre plane of a pillar. The testing revealed that:

- The load-bearing capacities of the pillar corners and, to a lesser extent, the pillar sides are small compared with those at the centre of the pillar. This highlights the influence of confinement on the strength of a pillar. The corners and sides are the least confined portions of the pillar whereas the central portion is subjected to the greatest confinement.
- The central portion of a coal pillar is capable of withstanding extremely high stress, even if the circumferential portions of the pillar have already failed.

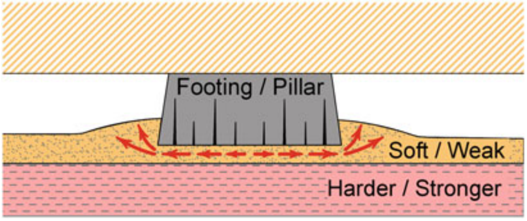


**Fig. 4.18** Bearing capacity failure and pillar deformation behaviour associated with extrusion of soft/weak foundation strata

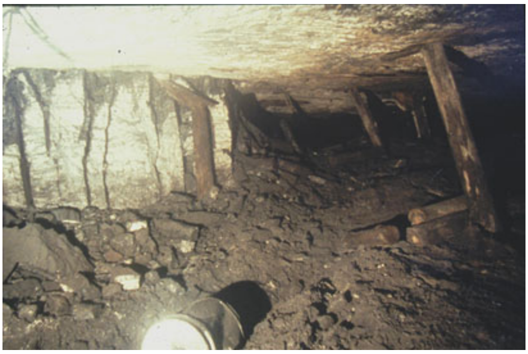


**Fig. 4.19** Extrusion of weak floor material from under a pillar, causing open vertical cracks to develop through the full width of the pillar under the influence of induced lateral tension

- The distribution of stress across a pillar is not uniform, especially once pillar load exceeds about two-thirds of the pillar strength. Hence, pillar design based on average pillar working stresses may need to make allowance for peak pillar stresses.

Figure 4.21 shows the development of lateral deformation, or dilation, in an in situ pillar monitored for microseismic activity and loaded to destruction in compression (Wagner 1974). Pillar dilation commenced at a stress level of about one-half of the ultimate strength of the pillar, with the rate of dilation reaching a maximum just prior to the peak load carrying capacity being exceeded. The microseismic activity is indicative of internal fracturing, with a remarkably close relationship being recorded between lateral deformation and frequency of internal fracturing, as shown in Fig. 4.21.

The approach to providing a mechanistic explanation of the effect of width-to-height ratio on pillar strength is usually referred to as the confined core concept (see, for example, Salamon 1992c). Wilson (1972) hypothesised that a pillar comprised an elastic core confined by yielding coal. Subsequently, Abel and Hoskins (1976), Barron (1984, 1986), Salamon (1992c, 1995b), Quinteiro (1993), and others have endeavoured to address a number of fundamental physical limitations in Wilson's hypothesis but a full mechanistic understanding is yet to be developed. In uncompleted research in the later part of his career, Salamon deduced that circumstances could arise where there were up to four distinct zones of behaviour within a coal pillar, shown in Fig. 4.22 and discussed in more detail in Sect. 4.4.5.6. It can be concluded on the basis of the confined core concept that:

- One effect of driving cut-throughs in a long pillar is to reduce the confinement provided to the core of the pillar in the longitudinal direction. Hence, a square pillar might be weaker than a rectangular pillar of the same minimum width.
- Designing a coal pillar not to fail provides no assurance that the roadways surrounding the pillar will be in a serviceable condition.
- Conversely, the physical state of the roadways surrounding a pillar is not necessarily a measure of the stability of the pillar.

#### 4.4.3.2 Effective Pillar Width

Rectangular and rhomboidal shaped pillars are an increasingly common feature of modern coal mining layouts. The traditional approach to the design of parallelepiped shaped pillars has been to base the prediction of their strength on the **minimum pillar width**, $w_{min}$, measured at right angles to the long side of the pillar, Fig. 4.4. Hence:

$$w_{\min} = w_1 \sin \theta \qquad (4.10)$$

As this approach may underestimate the strength of rectangular shaped pillars, some researchers have proposed that pillar strength be based on the concept of **effective pillar width**, $w_{eff}$. The most basic proposal is to equate the effective width to the square root of the area of the pillar, as per Eq. 4.10.

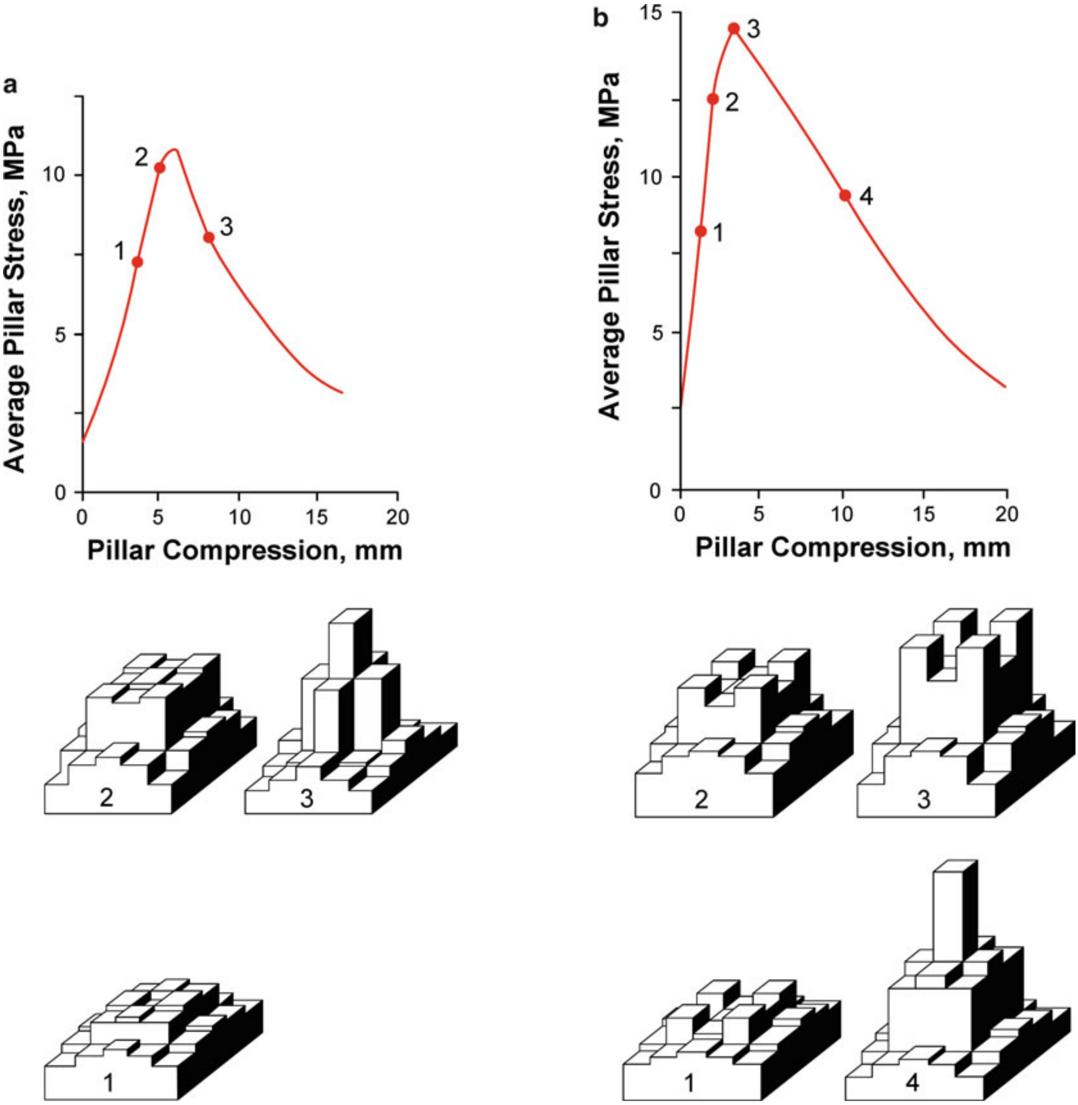


**Fig. 4.20** Stress profiles within in situ pillars at various stages of loading to destruction (After Wagner 1974). (**a**) w/h = 1, (**b**) w/h = 2

$$w_{eff} = \sqrt{w_1 \sin\theta\, w_2} = \sqrt{w_m w_2} \qquad (4.11)$$

Table 4.2 shows that this approach produces an unrealistic effective pillar width for long, narrow pillars. In reality, failure will progress from the narrow side of the pillar through to the pillar core well before benefits are fully realised from the extra confinement provided in the longitudinal direction.

Wagner (1974) proposed that the engineering concept of hydraulic radius could be applied to the calculation of effective width in the manner given by Eq. 4.12.

$$w_{eff} = 4\frac{A_p}{C_p} \qquad (4.12)$$

where

$A_p$ = *cross-sectional area of pillar*
$C_p$ = *circumference of pillar*

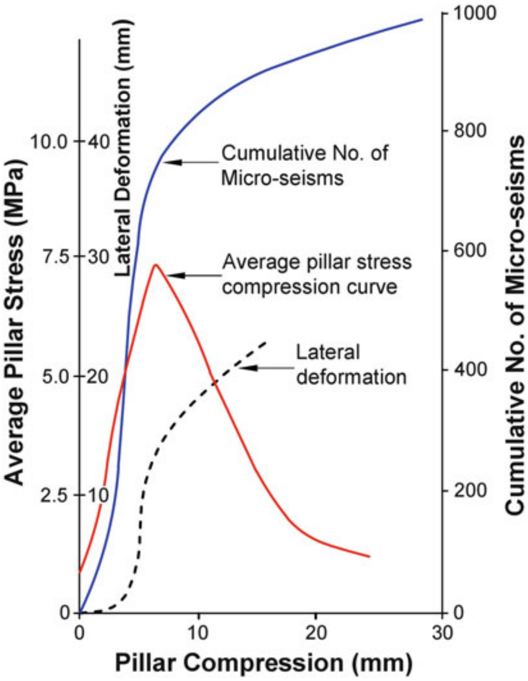


**Fig. 4.21** Behaviour of in situ pillars during compression (After Wagner 1974)

Application of Eq. 4.12 produces similar effective pillar widths to those of Eq. 4.11 when $w_{min}$ is greater than about 0.5 $w_2$ (Table 4.2). At moderate to small values of minimum pillar width ($0.2\ w_2 \leq w_{min} \leq 0.4\ w_2$), Eq. 4.12 predicts a smaller effective width, which is more sensible from a mechanistic viewpoint. However, at very small values of $w_{min}$ ($w_{min} < 0.2\ w_2$), the equation is still likely to overestimate the effective pillar width of narrow rectangular pillars.

Salamon et al. (1996) proposed a general solution to the calculation of the effective pillar width of a parallelepiped pillar by rewriting Eq. 4.12 as:

$$w_{eff} = \frac{4(w_m \times w_2)}{2(w_1 + w_2)} = \frac{2w_2 w_{\min}}{(w_1 + w_2)} \qquad (4.13)$$

$$= \left[\frac{2w_2}{w_1 + w_2}\right] w_{min} = \Theta_o w_{\min} \qquad (4.14)$$

The range for $\Theta_o$ is $1 \leq \Theta_o \leq 2$. $\Theta_o$ equals 1 if a pillar is square, rising towards a maximum value of 2 for an infinitely long pillar. Any benefits gained from confinement generated in the longitudinal direction can be expected to ramp up and plateau as the minimum pillar width increases, assuming a constant pillar height. The stage at which these benefits begin to materialise can be deduced to be a function of mining height; the lower the mining height, the less likely that failure will make its way through to the core across the minimum width of the pillar before the benefits of the additional longitudinal confinement start to be realised.

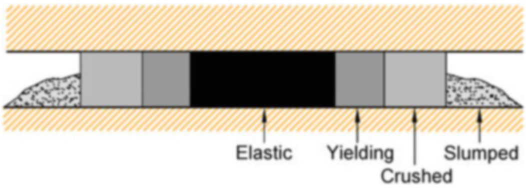


**Fig. 4.22** Zones developed within a coal pillar (Adapted from Quinteiro et al. 1995)

The proposal of Salamon et al. (1996) can be summarised as:

- Let $R_{min}$ = the minimum pillar width-to-height ratio, $w_{min}/h$.
- Let $R_l$ = the width-to-height ratio at which the strength benefits due to the greater longitudinal dimension first start to materialise.
- Let $R_u$ = the width-to-height ratio at which the full benefits due to the greater longitudinal dimension are realised.

If $R_{min} < R_l$, then set:

$$w_{eff} = w_{\min} = w_1 \sin\theta \qquad (4.15)$$

If $R_l \leq R_{min} \leq R_u$, then set:

$$w_{eff} = w_{\min}\Theta_o^{\frac{R_m - R_l}{R_u - R_l}} = w_{\min}\Theta \qquad (4.16)$$

If $R_{min} > R_u$, then set:

$$w_{eff} = w_{\min}\Theta_o = w_{\min}\left[\frac{2w_2}{w_1 + w_2}\right] \qquad (4.17)$$

The choice of the limiting width-to-height ratios, $R_l$ and $R_u$, is open to judgement, with Salamon et al. (1996) choosing:

$$R_l = 3 \qquad R_u = 6$$

The outcomes when these values are applied to a 100 m long rectangular pillar are also presented in Table 4.2.

**Table 4.2** Outcomes of applying various effective pillar width formulae to a 3 m high, 100 m long, rectangular shaped pillar

| | | | $w_{eff}$ | | |
|---|---|---|---|---|---|
| | | | $\sqrt{\text{Area}}$ | Hydraulic radius | Salamon et al. (1996) |
| $w_{min}$ | $w_2$ | h | Eq. 4.11 | Eq. 4.12 | Eqs. 4.15, 4.16, and 4.17 |
| 100 | 100 | 3 | 100.0 | 100 | 100 |
| 80 | 100 | 3 | 89.4 | 88.9 | 88.9 |
| 50 | 100 | 3 | 70.7 | 66.7 | 66.7 |
| 30 | 100 | 3 | 54.7 | 46.2 | 46.2 |
| 20 | 100 | 3 | 44.7 | 33.3 | 33.3 |
| 15 | 100 | 3 | 38.7 | 26.1 | 21.7 |
| 10 | 100 | 3 | 31.6 | 18.2 | 10.7 |
| 1 | 100 | 3 | 10 | 2.0 | 1 |

## 4.4.4 Scale Factors

In addition to width-to-height ratio, the strength of a coal pillar is a function of its size and volume. Laboratory testing of small and large scale coal specimens, such as reported by Bieniawski (1969) and Singh (1981), suggests that the uniaxial compressive strength of coal remains fairly constant beyond a cube size of one metre, as illustrated in Fig. 2.17, and that this strength is typically only 10–20 % of that of standard size laboratory specimens. In practice, mining height tends to be relatively constant over large areas within a coal seam and so variations in the shape and volume of coal pillars usually result from changes in their side dimensions (width, length) and, therefore, width-to-height ratio. The reader is referred to Sect. 2.6.2 for a more detailed discussion on the effect of size.

## 4.4.5 Determining Pillar Strength

### 4.4.5.1 Approaches

Numerous investigations dating back over a century have been undertaken into determining the strength of coal pillars. In general, pillar design procedures have their origins in one or a combination of the following approaches:

- Experiential
- Laboratory testing
- In situ testing
- Back-analysis
- Classical analytical methods
- Numerical modelling

### 4.4.5.2 Experiential

A number of minimum pillar size criteria have evolved from trial and error over the past two centuries. From a technical perspective, the in situ mining environment constitutes the best laboratory for evaluating pillar performance and these criteria can encapsulate valuable learnings. However, due to the high risks associated with unstable designs, trial and error is no longer acceptable.

Pillar design criteria based on field experience have met with mixed success. For example, back-analysis by Galvin (1992) of pillar size requirements legislated in NSW revealed that the probability of pillar failure in competent roof and floor conditions ranged from less than 1 in 10 million to as high as 6 in 10, depending on depth and mining height. The legislation was changed in 1982 to require a minimum pillar width of one-tenth depth or 10 m, whichever was the greater, with no regard to mining height. Following a serious, sudden collapse of bord and pillar workings that ranged in height from 6 to 10 m, the legislation was amended to require mine operators to seek approval to mine at heights greater than 4 m.

The stipulation of a minimum pillar width of one-tenth depth was based on British coal mining experience. Back-analysis of this simple criterion and the requirement for a minimum pillar width of 10 m reveals that it generally results in acceptable to conservative (more safe) outcomes by today's standards at all depths of mining when mining height is less than 3.5 m. However, caution is required at depths of less than 200 m when mining height exceeds 3.5 m. Caution is required in all circumstances when the immediate roof or floor has a low bearing capacity and when low shear strength interfaces are present in the pillar system. There are good reasons for retaining the requirement of a minimum pillar width of 10 m in many situations, regardless of the advent of more mechanistic approaches to pillar design.

#### 4.4.5.3 Laboratory Testing

Laboratory testing has found extensive application in deriving pillar strength formulae, with some of the better-known formulations summarised by Madden (1990). These generally relate to square pillars and take one of two forms given by Eqs. 4.18 and 4.19.

$$\sigma_{ps} = k_1\left\{r + (1-r)\left[\frac{w}{h}\right]\right\} \tag{4.18}$$

and

$$\sigma_{ps} = k_2\left[\frac{w}{w_o}\right]^{\alpha}\left[\frac{h}{h_o}\right]^{\beta} \tag{4.19}$$

where

$\sigma_{ps}$ = *pillar strength*
$k_1$ = *compressive strength of a cube of coal*
$k_2$ = *strength of a reference body of volume* $w_o^2 h_o$
$w$ = *width of pillar*
$w_o$ = *width of reference pillar*
$h$ = *height of pillar*
$h_o$ = *height of reference pillar*
$r$ = *a dimensionless constant*
$\alpha$ = *dimensionless width parameter*
$\beta$ = *dimensionless height parameter*

Formulae of the form of Eq. 4.18 are referred to as **linear pillar strength formulae** because they predict that pillar strength increases linearly in direct proportion to the width-to-height ratio of the pillar. Therefore, geometrically similar pillars are calculated to have the same strength, regardless of their actual dimensions. Formulae of the form of Eq. 4.19 are referred to as **power pillar strength formulae**. The reference body for this type of formulation is usually taken to be a cube of unit volume, so that $w_o$ and $h_o$ are both unity and can be omitted from the formulae. As shown by Salamon and Munro (1967), Eq. 4.19 can then be written as:

$$\sigma_{ps} = k_2 w^{\alpha} h^{\beta} \tag{4.20}$$

$$= k_2 V^a R^b \tag{4.21}$$

where

$R = w/h$ = *width to height ratio*
$V = w^2 h$ = *pillar volume*
$a = 1/3\,(\alpha + \beta)$
$b = 1/3\,(\alpha - 2\beta)$

Equation 4.21 illustrates that power formulations predict that pillar strength will increase exponentially with increase in pillar width-to-height ratio and, for a given width-to-height ratio, decrease with increase in pillar volume (except when $\alpha = \beta$, in which case pillar strength is independent of volume).

Shortcomings associated with basing coal pillar strength on laboratory testing include:

- Laboratory strength values for coal are typically up to seven times greater than those in the field, with one reason being that the strength of coal is size dependent (see Sect. 2.6.2).
- Loading rates in the laboratory are orders of magnitude faster than in the field. Because the strength of rock can reduce over time, this also contributes to coal strength being overestimated.
- The end conditions associated with samples subjected to laboratory testing do not reflect those in the field.
- There is a natural tendency for stronger portions of a coal seam to survive the sample collection, transportation and preparation process. Hence, laboratory specimens may not be representative of the overall coal mass.

- Coal seams usually comprise a number of coal bands of differing properties. It is difficult to characterise all of these bands to produce an overall pillar strength.

End conditions have been shown to have a significant effect on laboratory determined strength. Evans et al. (1961) found that lubrication of the faces of the loading platens could result in a 30 % reduction in the compressive strength of small cubes of coal. Strength reductions of 30–40 % were measured by Meikle and Holland (1965), who found that model pillar strength was in general agreement with the power pillar strength formula $\sigma_{ps} = k_2\sqrt{w/h}$ and that the material strength component, $k_{2,}$ was directly dependent on the condition of interfacial friction at the platens. The research outcomes of Brown and Gonana (1974), presented in Fig. 2.34, showed that when lateral end confinement was removed completely, neither the strength nor the post-failure stiffness of marble specimens increased with increasing width-to-height ratio. Peng (1978) reported a variation in the strength of laboratory tested samples of as much as 100 %, depending on end conditions. Similar results were obtained by Wagner (1980) when a thin lead sheath was placed between the end of a sample and the testing platen. This was also observed to change the failure mode to one of induced lateral tension, similar to that observed in the field and illustrated in Fig. 4.14.

Mark and Barton (1996) suggested that strength values obtained from the laboratory testing of coal cannot be utilised in a meaningful way for determining pillar strength. Other investigators have come close to making similar statements (for example, Mark (1990), Salamon (1991a), Galvin et al. (1994)). Subsequently, Medhurst and Brown (1996, 1998) conducted triaxial testing of large scale cylindrical coal specimens with a width-to-height ratio of 0.5, so as to avoid the influence of platen end effects in determining the material strength and deformability properties of coal. The Hoek-Brown failure criterion was then invoked to estimate the in situ strength of coal, with Medhurst and Brown (1998) concluding that the research provided evidence that the prudent choice of laboratory test method and associated analysis can lead to reliable estimates of seam strength. Brown (2014) advises that this approach in conjunction with analytical or numerical modelling to assess the effect of in situ end conditions provides a basis for estimating the strength of coal pillars.

In summary, a range of complexities and shortcomings are associated with basing pillar strength on laboratory testing. Nevertheless, laboratory testing can be very useful for conducting parametric and comparative studies. If undertaken prudently, it may provide a basis, in conjunction with analytical techniques and numerical modelling, for estimating coal pillar strength.

#### 4.4.5.4 In Situ Testing

In situ testing of coal pillars dates back to 1939 and has been undertaken in an attempt to overcome some of the limitations associated with laboratory testing, These tests have been restricted to a maximum pillar width of about 2 m, due primarily to difficulties in generating sufficient load to fail larger size pillars. Madden (1990) provides a summary of formulae developed from these testing programs.

The first comprehensive in situ tests were undertaken in South Africa by Bieniawski in the late 1960s. These involved cutting a slot at the top of the pillar, constraining the free end with either a strap, wood shuttering, steel shuttering or a reinforced concrete cap extending over the specimen sides (Fig. 4.23), and loading the pillar with hydraulic jacks connected to a common supply circuit (Bieniawski 1968; Bieniawski and van Heerden 1975). Based on the test results, Bieniawski proposed the linear formula defined by Eq. 4.22 for computing the in situ strength of square coal pillars in South Africa (Bieniawski 1967, 1987).

$$\begin{aligned}\sigma_{ps} &= 400 + 220\left[\frac{w}{h}\right] \qquad (psi)\\ &= 620\left\{0.645 + 0.355\left[\frac{w}{h}\right]\right\} \quad (psi)\\ &= 4.27\left\{0.645 + 0.355\left[\frac{w}{h}\right]\right\} \quad (MPa)\end{aligned} \tag{4.22}$$

Bieniawski considered this formula to be applicable only to pillars wider than a critical value of about 1 m and with a width-to-height ratio greater than unity. He was of the view that it

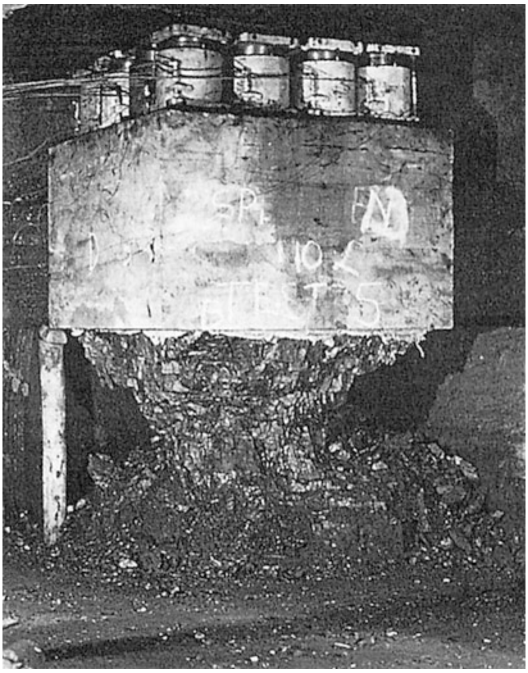


**Fig. 4.23** Load controlled in situ pillar testing in which the pillars were loaded at the roof/pillar contact (Courtesy of Professor Z.T. Bieniawski)

would underestimate the strength of pillars with a width-to-height ratio greater than 10.

Subsequently, it was reasoned that this method of testing pillars in situ:

- did not reflect the principal stress distribution that normally applied at the roof/pillar interface;
- destroyed a critical component that determined pillar strength, namely the natural end conditions; and
- did not mirror the true loading system, which was displacement controlled rather than load controlled.

Wagner (1974) addressed these deficiencies by, firstly, cutting a horizontal slot through coal pillars at their mid-height (a plane of symmetry) so as not to interfere with stress distributions and roof and floor contact conditions, and, secondly, by loading the pillars with uniform increments of displacement (mirroring roof convergence), Fig. 4.24. The strength of the test pillars was found to be about 50 % greater than that predicted from back-analysis by Salamon and Munro (1967). This was attributed to the in situ test loading rates being much higher than those operating under field conditions (Wagner 2010). Wagner's results continue to find wide application in the study of pillar confinement, pillar failure modes and post-failure stiffness and behaviour.

#### 4.4.5.5 Back-Analysis of Field Performance

Pillar design procedures developed from back-analysis of field performance have found the widest application in practice. Salamon and Munro (1967) reasoned that the statistical back-analysis of pillar performance provided the most reliable method of predicting full-scale pillar strength. Their approach, described by Salamon (1983) as semi-empirical, aimed to rationalise the accumulated experience of miners and was based on the belief that mine planners, over many decades of trial and error, did get quite close to the correct solution (Salamon 1999). However, the mine planners were unable to quantify their accumulated knowledge and so they passed on their experience largely through examples of their design.

Salamon and Munro (1967) adopted an empirical research approach to determining the strength of coal pillars in South Africa by focusing their analysis on the primary variables of coal strength, pillar width and pillar height. The effects of the remaining variables were taken into account by assigning a level of confidence to the design outcomes. Their research was concerned primarily with pillar strength in bord and pillar mining and was premised on:

- field data sourced only from mining panels where it could be ascertained with reasonable certainty that failure only involved the coal pillar element and was not associated with failure of the surrounding strata;
- panels associated with reasonably uniform geology and free of major structural disturbances;
- the mining layout in the relevant panels being, within reason, uniform;

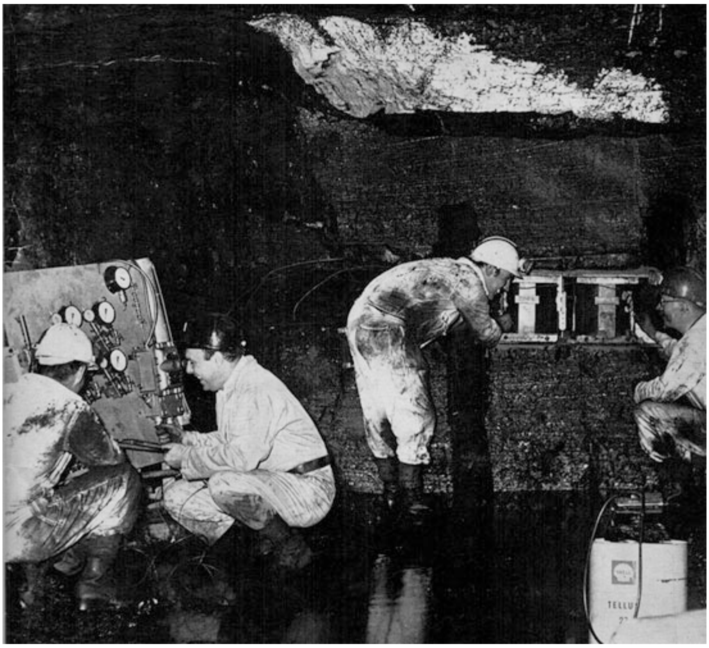


**Fig. 4.24** Displacement controlled in situ pillar testing as reported by Wagner (1974) in which the pillars were loaded at mid-height so as not to interfere with end contact conditions – l. to r. Neville Cook, Miklos Salamon, Keith Hodgson, Horst Wagner (After Chamber of Mines of South Africa 1969)

- field data only from panels that had a diameter at least equal to depth;
- pillar collapses that did not occur immediately after the formation of the pillars.

The researchers used these criteria as the basis for assuming that all pillars were loaded equally and in accordance with tributary area theory and for invoking a linear or power law pillar strength formula because the coal pillar was assumed to be the weakest component in the pillar system. Computational limitations of the day resulted in pillar strength being defined by the power relationship, given by Eq. 4.19 (Salamon 1995a). It was considered that, provided pillar working load could be approximated reasonably accurately by applying tributary area theory, pillar strength could be determined by statistically manipulating the values of $k_2$, α and β in this equation until the predicted pillar strengths best approximated the assumed pillar working stresses.

Salamon and Munro utilised the **maximum likelihood function** statistical method for this purpose because, importantly, it gave a weighting to the valuable information contained in successful outcomes (stable pillars) as well as to unsuccessful outcomes (collapsed pillars).

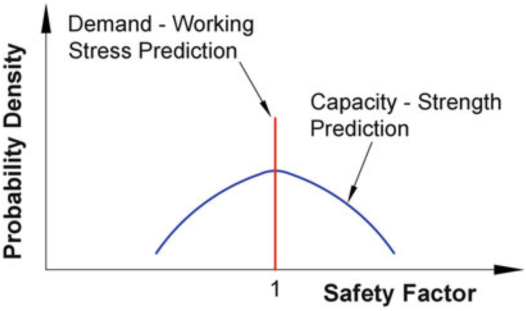


**Fig. 4.25** Conceptual probability density distributions applying to maximum likelihood statistical derivation of pillar strength by Salamon and Munro (1967)

Those values that maximise the sample likelihood are known as the **maximum likelihood estimates**. By assuming that pillar load at failure was known precisely, the probability distribution for load could be described by a straight line at a safety factor value of 1, Fig. 4.25. The statistical analysis then reduced to determining values for $k_2$, α and β that gave the tightest distribution of strength values about this line, resulting in the strength of a coal pillar under South African conditions being given by Eq. 4.23:

$$\sigma_{ps} = 7.2\,\frac{w^{0.46}}{h^{0.66}}\,(MPa) \tag{4.23}$$

Equation 4.23 was derived from a field database covering a width-to-height ratio range of 0.9–3.7 for collapsed cases and 1.2–8.8 for stable cases. Salamon and Oravecz (1976) reported that the equation underestimated the strength of pillars with a width-to-height ratio greater than 5 or 6 and that there was some evidence that when pillars have a width-to-height ratio exceeding, say 10, they do not fail under any practically possible load. In 1982, Salamon suggested Eq. 4.24 as an extension to Eq. 4.21 to account for the increased strength of so-called squat pillars (Salamon et al. 1996).

$$\sigma_{ps} = k_2 V^a R_o^b \left\{\frac{b}{\varepsilon}\left[\left(\frac{R_{\min}}{R_o}\right)^{\varepsilon} - 1\right] + 1\right\} \tag{4.24}$$

where

$V$ $= w_{\min}^2 h$

$R_{\min}$ = *minimum width to height ratio* $w_{min}/h$, *of a pillar*

$R_o$ = *the width to height ratio at which a pillar is considered to become squat*

$\varepsilon$ = *a measure of the rate of strength increase once* $w_{min}/h$ *exceeds* $R_o$

An extensive evaluation of this formula in the laboratory and the field was undertaken by Madden (1990). It is now applied extensively in South Africa on the basis that $R_o = 5$ and $\varepsilon = 2.5$.

Salamon et al. (1996) undertook a range of statistical analyses of an Australian database comprising unstable cases with a width-to-height range of 1.1–8.2 and stable cases in a width-to-height range of 1.7–11.2. This analysis was based on effective pillar width and squat pillar behaviour and produced the so-called **UNSW Pillar Strength Formulae,** given by Eqs. 4.25, 4.26, and 4.27.

*Linear* :

$$\sigma_{ps} = 5.12\left\{0.56 + 0.44\left[\frac{w_{\min}}{h}\right]\right\} \quad (MPa) \tag{4.25}$$

(Note: The linear formula is premised on minimum pillar width only)

*Power* :

$$\frac{w_{\min}}{h} < 5: \qquad \sigma_{ps} = 8.60\,\frac{(w_{\min}\,\Theta)^{0.51}}{h^{0.84}} \quad (MPa) \tag{4.26}$$

$$\frac{w_{\min}}{h} \geq 5: \qquad \sigma_{ps} = \frac{27.63\,\Theta^{0.51}}{w_{\min}^{0.220}\,h^{0.110}}\left\{0.290\left[\left(\frac{w_{\min}}{5h}\right)^{2.5} - 1\right] + 1\right\}(MPa) \tag{4.27}$$

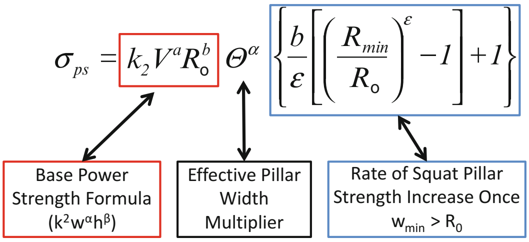
where

$$For\ \frac{w_{\min}}{h} < 3,\ \Theta = 1$$

$$For\ 3 \leq \frac{w_{\min}}{h} \leq 6,\ \ \Theta = \left[\frac{2w_2}{w_1 + w_2}\right]^{\frac{\frac{w_{\min}}{h} - 3}{3}}$$

$$For\ \frac{w_{\min}}{h} > 6, \Theta = \left[\frac{2w_2}{w_1 + w_2}\right]$$

Equation 4.27 represents the most general form of a power squat pillar formula and is comprised of the following components:


$\sigma_{ps} = \boxed{k_2 V^a R_o^b}\ \Theta^{\alpha}\ \boxed{\left\{\frac{b}{\varepsilon}\left[\left(\frac{R_{min}}{R_o}\right)^{\varepsilon} - 1\right] + 1\right\}}$
Base Power Strength Formula ($k^2 w^{\alpha} h^{\beta}$)
Effective Pillar Width Multiplier
Rate of Squat Pillar Strength Increase Once $w_{min} > R_0$

The UNSW pillar strength analysis also revealed that the underlying pillar strengths in South Africa and Australia resembled each other closely. This outcome is not inconsistent with the findings of Evans and Pomeroy (1973) that the strength of almost all coals are virtually indistinguishable at high confining pressures.

In the early 1980s, Bieniawski applied his linear pillar strength formula developed for the Witbank coalfield in South Africa (Eq. 4.22) to bord and pillar operations in the USA (Bieniawski 1983). Analysis of a database of some 200 panels of pillars resulted in the pillar strength formula defined by Eq. 4.28 being recommended.

$$\sigma_{ps} = k_1\left\{0.64 + 0.36\left[\frac{w_1}{h}\right]\right\}(MPa) \tag{4.28}$$

where

$k_1$ = *Scaled up strength of a cubical laboratory specimen*
*Note* : $S_1$ *and* $\sigma_1$ *are used in some publications in place of* $k_1$.

This formula, referred to as the **Bieniawski formula**, is premised on a view that pillar volume has no further impact on pillar strength once pillar width is greater than 1 m (assuming pillar height is also ≥1 m). Therefore, $k_1$ represents the strength of a 1 m coal cube. This strength was determined to be 6.2 MPa for the Pittsburgh coal seam using a scaling formula proposed by Hustrulid (1976), thus resulting in the following pillar strength formula.

$$\sigma_{ps} = 6.4\left\{0.64 + 0.36\left[\frac{w_1}{h}\right]\right\}(MPa) \quad (4.29)$$

This formula is often referred to as the **Bieniawski-PSU formula** and has found application in a range of international settings.

Subsequently, the **Mark-Bieniawski formula** for the strength of rectangular shaped pillars, given by Eq. 4.30, was developed out of this formula on the assumption that the stress within the yield zone of a pillar is a continuous function of the distance from the nearest rib (Mark et al. 1988).

$$\sigma_{ps} = k_1\left[0.64 + 0.54\left(\frac{w_1}{h}\right) - 0.18\left(\frac{{w_1}^2}{w_2 h}\right)\right](MPa) \quad (4.30)$$

It is noteworthy that Wilson (1972) calculated analytically that if 10 ft (~3 m) wide cut-throughs were driven at 100 ft (~30 m) centres in a 3000 ft (~900 m) long by 100 ft (~30 m) wide by 10 ft (~3 m) high pillar, the strength of the pillar would be reduced by 28 % due to the introduction of extra yield zones. When applied to the same example, the Mark-Bieniawski formula predicts a 32.4 % reduction in pillar strength and the UNSW power formula predicts a 37.1 % reduction.

Outcomes from the back-analysis of coal pillar field performance now feature in a range of hybrid pillar design procedures. For example, Wagner (2003) applied the Salamon and Munro formula (Eq. 4.23) to hard rock mining situations by using the Hoek-Brown rock mass strength criterion to estimate $k_2$. Malan and Napier (2011) report that the Salamon and Munro formula has also been applied extensively in South African chrome mines using a $k_2$ value of between one-third and two-thirds of the UCS of the pillar material.

### 4.4.5.6 Analytical Methods

The analytical approach to determining pillar strength is founded on principles of physics and applied mechanics and recognises the role of the surrounding strata in determining pillar stress and deformation. This approach is fundamental to developing a sound mechanistic understanding of the total pillar system and underpins sensible numerical modelling. Although considerable progress has been achieved, it remains a work in progress.

Prior to a pillar beginning to yield, the stresses are highest at the pillar edges. Once these exceed coal strength and yielding commences, the edges must shed load, transferring it further into the pillar. If the adjacent coal is sufficiently confined, it will be able to accept the additional load. Otherwise, the simultaneous yielding and load transfer will proceed further inwards until either sufficient confinement is generated to establish a new state of equilibrium or, if this proves impossible, the pillar fails.

It follows, therefore, that for increasing squatter pillars to have a higher strength, their increased width-to-height ratio must be causing an increase in confinement to the core of the pillar. For this to occur, frictional resistance has to be generated between the yielded coal and the roof and floor of the mining horizon to restrain the migration of this coal as the pillar dilates. The magnitude of this restraint and the associated horizontal stress grow with increasing distance into the pillar. This has to be balanced by horizontal stress induced in the immediate roof and floor strata surrounding the pillar, which can impact adversely on the stability of this strata. This description of pillar behaviour is the basis of the so-called **confined core concept**, with a more detailed analysis of the underpinning process provided by Salamon (1992c). The confined core concept provides the physics and applied mechanics foundations of the general solution for pillar behaviour and failure modes.

One of the better known approaches to developing the confined core concept is Wilson's hypothesis (Wilson 1972; Wilson 1977; Carr and Wilson 1983). Wilson divided a coal pillar into a central core subjected to triaxial stress conditions and a surrounding yield zone that constrained the inner core. Approximate rules were developed to describe the state of stress in each of these zones and in the surrounding goaf, taking into account different roof conditions.

Wilson's hypothesis provided considerable insight, impetus and direction to pillar mechanics and is widely recognised as the first rational attempt to design longwall chain pillars. However, in order to develop the hypothesis, Wilson had to make a number of sweeping assumptions, many of which have been shown to have tenuous physical bases. Subsequently, the confined core concept has undergone further development, most notably by Abel and Hoskins (1976), Barron (1983, 1986), Barron and Pen (1992), Salamon (1992a, c), Quinteiro (1993) and Salamon (1995b). Nevertheless, the concept is still constrained by a number of shortcomings.

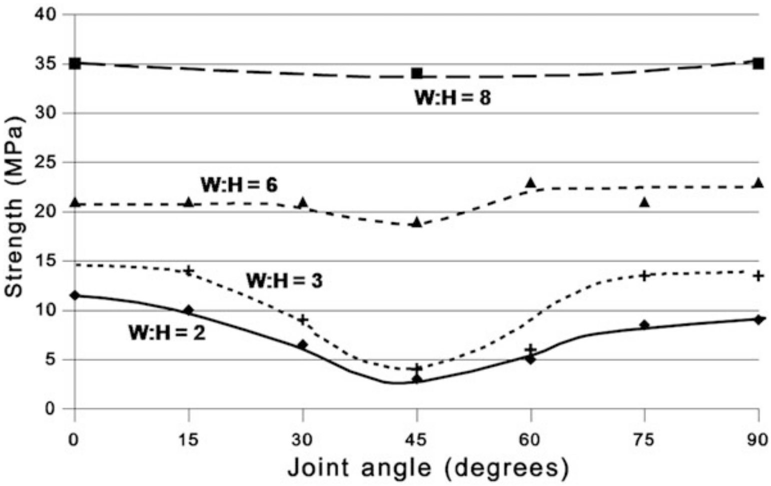
Classical analytical solutions can give valuable insight into the elements that govern the behaviour mechanics of pillar systems. This is evidenced, for example, by the research findings of Salamon (1995b) discussed in Sect. 4.6.3 in relation to the concept of a confined core. However, their application has been restricted by a lack of computational power to solve the associated mathematical relationships and an inability to analyse the interrelationships between all the various mechanisms that determine pillar system response to mining. The advent of powerful computing systems is now aiding in addressing these limitations through numerical modelling.

#### 4.4.5.7 Numerical Modelling

Numerical modelling finds a wide range of applications in assessing pillar system behaviour and strength and, as with analytical techniques, can give valuable insight and direction to design and stability assessment despite an incomplete theoretical knowledge base. However, unreliable outcomes are often produced or the full potential of numerical modelling is not realised because the required input data does not exist or because the modeller has a poor understanding of the scope, limitations and operation of numerical modelling codes and techniques (see Sect. 2.7.3). Notwithstanding these factors, numerical modelling offers many benefits in coal pillar analysis and design arising from its capacity to:

- analyse coal pillar systems on a regional as well as a local basis;
- more accurately define the pre-mining environment, including the disposition, composition and geomechanical properties of strata, horizontal stress fields, joints, bedding planes and other geological structures, all of which have the potential to impact critically on coal pillar system performance;
- more accurately estimate pillar load in general and, in particular, in irregular mining layouts, by taking account of the stiffness of both the surrounding strata and the individual coal pillars;
- model the process of mining to incorporate mining-induced changes in stress fields, such as associated with the formation of goaves;
- analyse the behaviour of the roof/pillar/floor system as a composite structure;
- apply more sophisticated failure criteria and to undertake an assessment of the various constitutive laws that may apply to the given circumstances;
- simultaneously assess multiple potential failure modes e.g. shear fracture, tensile failure, bedding plane shear;
- quantify stress paths and track changes in failure modes;
- facilitate the evaluation of parameter variations on pillar system behaviour;
- identify the criticality of individual parameters on pillar system behaviour and performance;

**Fig. 4.26** Effect of joint inclination on pillar strength as determined from numerical modelling by Esterhuizen (1997)



- eliminate the guesswork in choosing the mechanism or mechanisms that may ultimately bring about failure;
- quantify the locations of failure, the effect of failure on stress redistribution and residual strength properties, and post-failure behaviour;
- simulate the effect of pillar system behaviour on roadway behaviour and vice versa; and
- incorporate time and loading rate effects on pillar system behaviour.

Empirical and classical analytical pillar strength prediction techniques, supported by field measurements, are important for calibrating numerical models and for validating their outcomes. Conversely, numerical models may provide an insight into the mechanics of the deformation process and allow a better interpretation of empirical and field measurement data within the geological constraints of the pillar system (Li et al. 2005). Numerical models can also assist in identifying and evaluating potential pillar behaviour modes in situations which fall outside existing experience and empirical databases.

Three notable examples of how numerical modelling techniques, supported by empirical analysis and field data, can be applied to evaluating coal pillar strength are those reported by Esterhuizen (1997), Gale (1998) and Watson (2003). Esterhuizen utilised the discrete element program, UDEC, to evaluate the effect of joint inclination on pillar strength as a function of pillar width-to-height ratio. This mathematical model was designed to simulate elastic and plastic deformation of an assembly of blocks in two dimensions, with the joint interfaces between the blocks being treated as Coulomb surfaces. The modelling method and input parameters were calibrated against the in situ pillar testing outcomes of Wagner (1974) and tested against another suite of empirical equations. Figure 4.26 summarises the modelling outcomes. It illustrates the insight that numerical modelling can give to a variable that, on the basis of engineering judgment, could be expected to impact on pillar strength but which falls outside the realm of existing experience and empirical databases.

Gale and Mills (1994) undertook two-dimensional numerical modelling utilising the finite difference code, FLAC2D, to simulate the effect of weak geological boundary conditions on the generation of confining stress at the ends of the pillars. The modelling produced a range of pillar strengths which were then correlated with field data from Australia and the USA as shown in Fig. 4.27.

Watson (2003) used the Salamon and Munro pillar strength formula (Eq. 4.23) to calibrate a numerical pillar strength prediction model based on the two-dimensional finite element code Phase$^2$. A three-dimensional elastoplastic analysis based on the finite element program PLAS3D was also calibrated to the Salamon squat pillar formula (Eq. 4.24). It was proposed that the

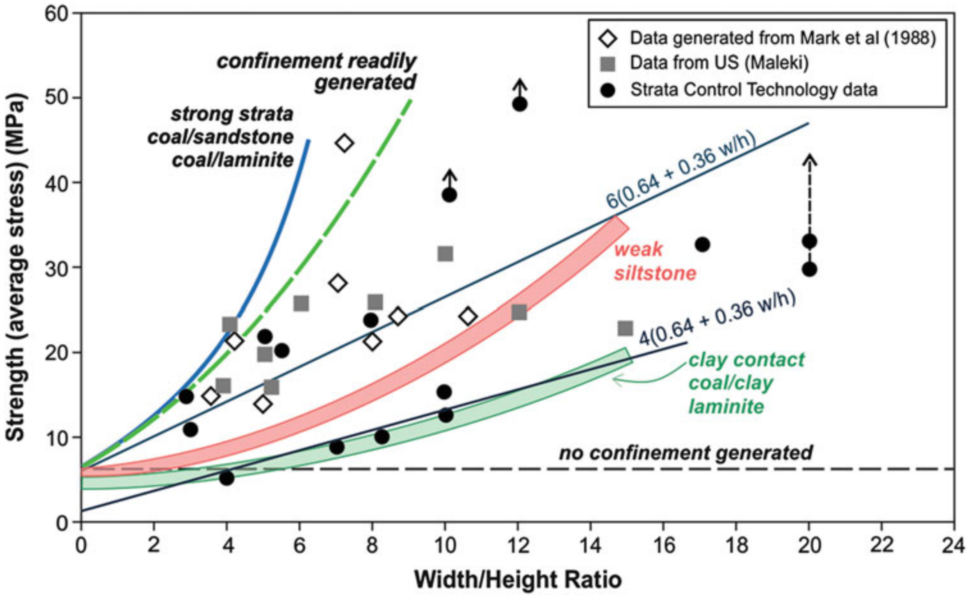


**Fig. 4.27** Effect of pillar end conditions on pillar strength as determined from numerical modelling by Gale and Mills (1994)

strength parameters produced by the models could be used as input into other finite element and finite difference programs.

These three examples illustrate the opportunities offered by numerical modelling for better understanding and designing coal pillar systems. It must always be recognised, however, that the validity of outcomes is dependent on the accuracy of the modelled geological features, material properties, constitutive laws, boundary conditions and stress paths. If these are not representative of the field conditions or if the model is not simulating mechanisms observed in practice, then the outcomes can be very misleading. Therefore, calibration and validation against field data, sensitive analysis of modelling outcomes to variations in input data, and experience are all important in assessing the reliability of modelling outcomes. The use of more sophisticated models should not be construed to mean that outcomes are more accurate or reliable.

Because coal pillars are loaded as a result of the deformation of the surrounding rock mass, it is essential that a numerical model describes the behaviour of coal measures fairly well. Surface subsidence is a response to the same deformation process and, therefore, comparisons between predicted and measured surface subsidence provide a check on the reliability of a numerical model of pillar behaviour.

#### 4.4.5.8 Summary Points

The plethora of approaches and outcomes relating to determining coal pillar strength is a reflection of the state of the knowledge relating to coal pillar behaviour; the variety of functions that coal pillars perform; the uncertainty associated with coal pillar design; and the need for end-users to have an understanding of the scope and fundamental principles associated with any design procedure they adopt. Some of the more important conclusions from these various approaches, which give direction to pillar design include:

- The in situ material strength of coal is typically in the range of 5–7.5 MPa.

- A minimum pillar width of one-tenth depth in working heights up to 3.5 m provides a useful first pass approximation of pillar size required for operational stability in situations where roof and floor strata failure are not associated with pillar failure.
- No single pillar design procedure caters for all mining circumstances and pillar functions.
- Linear and power pillar strength formulations provide a basis for pillar design in situations where roof and floor strata failure are not associated with pillar failure.
- The Salamon and Munro and UNSW pillar strength formulations do not apply to situations where roof and floor strata failure are associated with pillar failure or where behaviour is predominantly influenced by geological features, albeit that they can be useful aids for evaluating such situations.
- In critical situations where the consequences of pillar instability cannot be tolerated, consideration should be given to basing pillar design on the minimum pillar width.
- If diamond shaped pillars are employed in critical situations, consideration should be given to increasing their design safety factor in recognition of the propensity for spalling of acute pillar corners.
- The in situ pillar tests of Wagner (1974) remain one of, if not the only, point of reference for the post-failure stiffness of in situ pillars.
- Pillar system failure mode can change and pillar system strength can be reduced significantly in the presence of low cohesion and friction interfaces.
- While limitations are associated with numerical modelling, if undertaken sensibly it can provide valuable insight into predicting, understanding and validating pillar behaviour.
- Numerical modelling currently provides the most reliable basis, albeit incomplete, for endeavouring to quantify the impacts on pillar system performance of low cohesion and friction interfaces, soft or weak foundations and geological structure.
- Parametric and sensitivity analyses of pillar design input parameters are important risk management measures.

## 4.5 Quantifying Design Risk

### 4.5.1 Probabilistic Stability Prediction

The concept of safety factor as the traditional engineering approach for assigning a level of reliability to design outcomes; its advantages and disadvantages; and how these may be addressed by a probabilistic approach to quantifying variability and uncertainty in design are discussed in Sect. 2.7.4. A purely safety factor approach to pillar design relies on judgement and experience. The recommendations of Bieniawski (1983, 1992) regarding the application of the Bieniawski formula (Eq. 4.28) to USA coal mining conditions and summarised in Table 4.3 provide an example of this approach.

Bieniawski emphasised that these safety factor values should be used as a guide only and that local mining experience should be taken into consideration. Cheken and Listak (1994) recommended safety factors ranging from 1.5 to 2.2 when the formula is applied to multi-seam situations. The safety factors assigned to the use of the Bieniawski formula have not been correlated to probabilities of stability and instability.

Salamon and Munro (1967) developed a partial probabilistic approach to pillar design by using the maximum likelihood method to quantify the probability that their equation (Eq. 4.23) accurately predicted pillar strength when pillar system failure only involved the coal element. Some 18 years later, the formulation had been associated with the successful design of over 1.2 million coal pillars in South Africa (Salamon and Wagner 1985) and it continues to find widespread application in that country.

**Table 4.3** Summary of safety factor recommendations of Bieniawski (1983, 1992) when using the Bieniawski formula (Eq. 4.28)

| Situation | Safety factor |
|---|---|
| Bord and pillar first workings | 1.5 |
| Pillar extraction | 2.0 |
| Main development pillars | 2.0 |
| Barrier pillars | 2.5 |
| Tailgate chain pillars | 1.3 |
| Pillars in bleeder roadways | 1.5–2.0 |

**Table 4.4** Statistical confidence levels associated with UNSW pillar design formulae

| | UNSW linear formula (Eq. 4.25) | UNSW power formulae (Eqs. 4.26 and 4.27) |
|---|---|---|
| **Standard Deviation** | 0.207 | 0.157 |
| **Probability of Failure** | **Safety Factor** | |
| 8 in 10 | 0.84 | 0.87 |
| 5 in 10 | 1.00 | 1.00 |
| 1 in 10 | 1.30 | 1.22 |
| 5 in 100 | 1.40 | 1.30 |
| 2 in 100 | 1.53 | 1.38 |
| 1 in 100 | 1.62 | 1.44 |
| 1 in 1 000 | 1.85 | 1.63 |
| 1 in 10 000 | 2.09 | 1.79 |
| 1 in 100 000 | 2.42 | 1.95 |
| 1 in 1,000,000 | 2.68 | 2.11 |

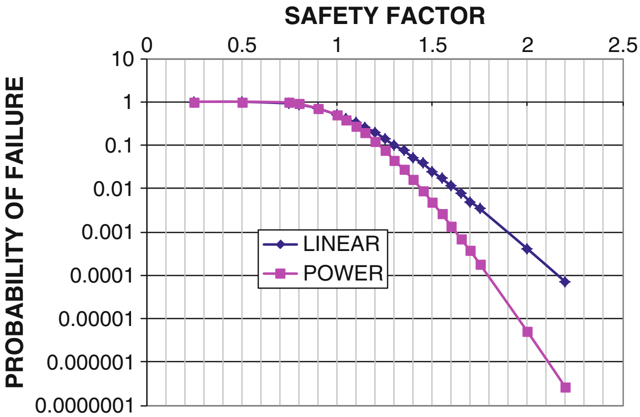


**Fig. 4.28** Safety factor versus probability of failure associated with UNSW pillar design formulae

The same approach was applied to developing the UNSW pillar strength formulae, producing the confidence levels presented in Table 4.4 and Fig. 4.28. The standard deviation for the UNSW linear pillar strength formula (Eq. 4.25) is greater than that for the UNSW power strength formulae (Eqs. 4.26 and 4.27), indicating that there is a lower confidence level associated with the linear formula. Hence, a higher safety factor is required to achieve the same level of confidence in the design outcomes as when using the power formulae.

The consequences associated with an unstable structure are the primary determinant in selecting a design probability. Consistent with risk management principles, the designed probability of success, or safety factor, should be increased in line with the consequences of structural instability.

Salamon and Munro (1967) determined that 50 % of stable pillar cases were most densely concentrated within a safety factor range of 1.31–1.88. They proposed that a safety factor of 1.6, corresponding to a probability of failure of about 1 in 650, would be acceptable in most single seam bord and pillar circumstances, subject to local experience (Salamon and Munro 1967). Subsequently, Salamon and Oravecz (1976) recommended a safety factor of 2 in circumstances where it was important that long term stability be maintained. If more than one seam is to be mined, they considered it advisable to require a safety factor somewhat greater than 2.

Figure 4.29 shows the histogram of safety factors for Australian failed pillar cases based on the UNSW power pillar strength formulae and tributary area load. The safety factors range from

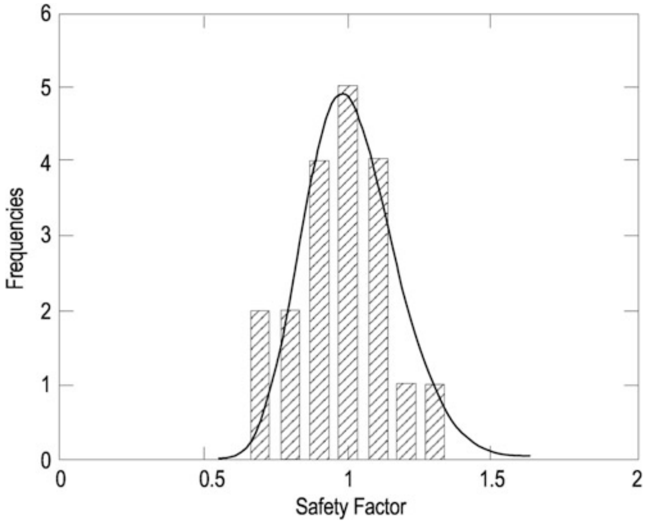


**Fig. 4.29** Histogram of frequency of failure versus safety factor constructed from the Australian database employing the UNSW power law pillar strength formulae and utilising the maximum likelihood method (After Salamon et al. 1996)

0.74 to 1.39. This scatter reflects the approximations, assumptions and potential errors in the input data associated with this methodology. These factors encapsulate the two types of uncertainty that need to be considered in probability assessments, namely **aleatory** and **epistemic**. Aleatory uncertainty is 'random' uncertainty or natural variability associated mainly with the high spatial variability of geologic material properties. Epistemic uncertainty is uncertainty due to model accuracy and lack of information about parametric estimation (Stewart and O'Rourke 2008). In the case of the UNSW pillar strength formulations, these uncertainties include:

- approximating gravitational acceleration to 10 m/s$^2$;
- approximating the effective overburden density to 2500 kg/m$^3$;
- assuming pillar strength to be a function of only three parameters, namely coal material strength, pillar width and pillar height;
- the assumed ramp up rate of effective pillar width;
- the assumed width-to height-ratio of transition to a squat pillar;
- the assumed rate of strength increase of squat pillars;
- assuming that all pillars in the database were subjected to full deadweight loading;
- computing pillar strength based on average pillar stress rather than the actual stress distribution within the pillar;
- natural variations in geological conditions and material property distributions between the sites from which the data was collected;
- variation in the effective load carrying area of the pillar edges associated with the excavation technique (e.g. hand mining, drill and blast, machine cutting);
- mining dimensions derived from plans which may not have accurately depicted the actual working dimensions; and
- a database that is likely to be biased towards undersized rather than oversized pillars.

The solid line shown in Fig. 4.29 has the shape of a normal or Gaussian distribution and is symmetrical about its mean. This is an assumed and purely theoretical distribution and one of a number of possible mathematical functions that could have been used to denote the shape of the distribution of safety factor versus frequency of failure (see Sect. 2.7.5). Salamon and Munro assumed that the logarithm (base 10) of the safety factors had a normal distribution profile about a mean safety factor of 1. The UNSW analysis mirrored this analysis but, for reasons of convenience, it was based on the natural logarithm. In both cases the resulting probability distribution function is referred to as a lognormal distribution. The power of this approach is that because it is based on back-analysis, it takes into account the variation of the size and shape of the pillars and, therefore, a stochastic modelling approach is not required to deal with the variation between the design and actual dimensions.

The type of assumed statistical distribution can affect the correlation between probability and safety factor and has a bearing on the estimation of tail probabilities, which are important if pillar design is to become fully probabilistic. Insufficient data hampered the evaluation of alternative distributions until 2006, when Salamon et al. (2006) repeated the analysis for a Weibull distribution and a Gamma distribution. The Weibull distribution produced significantly poorer quality outcomes than the lognormal distribution, while the Gamma distribution gave comparable qualities of fit and very similar fitting functions. The researchers concluded that the lognormal distribution gave reasonably robust and consistent results and, hence, it continues to underpin the statistical analysis.

Table 4.4 shows that increasing the safety factor beyond 1.63 when using the UNSW power pillar strength formulae and beyond 1.85 when using the UNSW linear pillar strength formula, only has the potential to reduce the probability of failure by 1 in 1000. Nevertheless, as the consequences of failure increase, lower probabilities of failure may be justified. It is not uncommon to design to probabilities of 1 in 1,000,000 or greater when the consequences are high. Often, percentage extraction only has to be reduced marginally to achieve orders of magnitude reduction in risk.

### 4.5.2 Probabilistic Design

Theoretically, the Salamon and Munro and the UNSW pillar design methodologies do not constitute a complete probabilistic approach to pillar design in competent roof and floor conditions. Firstly, they only quantify the risk associated with the ‘resisting’ component of the design, being the pillar strength, and, secondly, they do not incorporate an annualised probability of survival. Nevertheless, provided that the mining dimensions are known to a comparable or better level of accuracy to those used in deriving the pillar strength formulations, application of the associated pillar strength probabilities should produce a conservative outcome in other comparable situations. This is because care was taken to include only cases where mining dimensions were thought to be reasonably accurate and the ‘driving’ force was considered to be at a maximum (being deadweight loading).

The situation may be different in the case of old workings where there is a lack of confidence in the original ‘as-built’ mining dimensions and/or where the workings have deteriorated with the passage of time. These dimensional variations may fall well outside those that contributed to the probability distribution associated with the pillar strength determinations, in which case they constitute a source of epistemic uncertainty. This does not impact on the probability distributions associated with the pillar strength formulations but it does impact on the input values to these formulations and to the calculation of the driving force, being pillar load.

In a full probabilistic risk determination, this uncertainty is taken into account by assigning probability density functions to the mining dimensions (see Sect. 2.7.5). History attests that it is extremely unlikely in the case of old workings that these will be normal distributions or that the arithmetic mean of actual mining dimensions will lie close to the design or plan dimensions. Mining operations rarely extract less coal than shown on a plan; roof falls tend to occur to a consistent horizon so increasing effective pillar height; and roadway dimensions increase as pillars deteriorate.

Probabilistic based stability analysis is usually undertaken in the context of ‘annualised probability of failure’. The Salamon and Munro and the UNSW databases are too small to support this type of analysis. Hence, although a probability can be assigned to a pillar layout in terms of how many panels of pillars of a given layout are likely to fail, this probability cannot be related to a time scale. In attempting to address this limitation, Salamon et al. (1998) and van der Merwe (2003) postulated models that are based on assigning scaling (spalling) rates and computing whether failure will occur before pillar scaling is arrested by the surrounding bords becoming choked with rib (sidewall) spall. The models also estimate how long after pillar formation failure will occur. Van der Merwe has concluded that the majority of failures in South Africa have occurred within the first 10–20 years of pillar formation, with his model predicting an average pillar life of 44,000 years. It should be noted that neither approach gave consideration to flooding of the mine workings in the longer term which, depending on local conditions, can increase or decrease pillar stability (discussed in Sect. 11.7).

The findings regarding pillar survival in South Africa are not inconsistent with Australian experience. The age at failure of 16 of the 19 failed Australian cases studied by Salamon et al. (1996) was less than 20 years, with most failures occurring within the first year. The age at failure of the other three cases is not known, although one was at least 80 years old and, possibly up to 170 years old.

### 4.5.3 Summary Points

Some of the more important conclusions relating to quantifying uncertainty when designing pillars include:

- Not all pillar design procedures are currently able to be supported with performance probability profiles.
- Unless safety factor can be correlated to the probability of a successful design outcome, a safety factor of not less than 2 should be adopted if utilising Salamon and Munro or UNSW pillar strength formulations. This is not to say that a higher safety factor may not be required.
- The Salamon and Munro and UNSW pillar stability performance probabilities are specific to the assumptions and formulations associated with their derivation.
- The Salamon and Munro and UNSW pillar stability performance probabilities do not apply to situations where roof and floor strata failure are associated with pillar failure or where behaviour is predominantly influenced by geological features.
- The correlation between safety factor and performance probability reflects the level of confidence in a design procedure.
- The selection of a design safety factor should give careful consideration to the consequences of a pillar system failure.

## 4.6 Pillar Failure Modes

### 4.6.1 Types

In this text, the term **conventional failure mode** refers to those situations where a group or panel of pillars deform as an integrated system due to the load exceeding the strength of the pillars comprising the system. A failure mode involving an instantaneous release of strain energy stored within the rock fabric is referred to as a **pressure burst**, as distinct from an **outburst** which is a form of pressure burst associated with high gas concentrations. Discussion of pressure bursts and outbursts is deferred to Sects. 11.8 and 11.9, respectively. The term **dynamic confined core failure** has been coined to refer to the possible third mode of failure identified from the research of Salamon (1992c), Quinteiro (1993) and Salamon (1995b).

### 4.6.2 Conventional Failure Mode

Incidents in both the hard rock and soft rock mining sectors demonstrate that pillars in a bord and pillar mining system can fail in either a gradual or a sudden manner. Gradual failure is characterised by ample warning signs, such as rib or sidewall spall over an extended time period and can often be arrested by reinforcing the pillars. This failure mode is referred to as **controlled**

**failure** and includes those modes described colloquially as **creep** and **squeeze**. Sudden failure, on the other hand, may not be preceded by any deterioration in the condition of the pillars and, once initiated, cannot be arrested. Hence, it is referred to as **uncontrolled failure**. The very limited or total lack of warning of an impending uncontrolled failure and the speed at which it propagates present major risks of entrapment and windblast.

Once pillar working stress exceeds pillar strength, the mode of conventional failure is determined by the interactive behaviour of the surrounding strata and the pillars. The mechanism is analogous to that described in Sect. 2.6.11 for a rock specimen being loaded in a laboratory testing machine. The overburden load constitutes the testing machine and the coal pillars constitute the rock specimen.

An understanding of the mechanics governing whether failure develops in a controlled or an uncontrolled manner is aided by considering a long isolated panel that is widened in increments by forming more pillars. This process causes a progressive increase in the load acting on the pillars in the centre of the panel. The first critical stage in this process occurs if the load on these central pillars becomes equal to their strength. From that point onwards, the mining panel is only conditionally stable because the slopes of the load-deformation curves of the pillars in the centre of the panel are negative due to the pillars having exceeded their peak strength. The stiffness of the loading system, or overburden, determines the extent to which this load will follow the pillars as they yield. The pillars will continue to unload in a controlled manner while ever the stiffness of the superincumbent strata is greater than the post-peak stiffness of the pillars.

As the panel continues to be widened, the stiffness of the superincumbent strata trends towards zero. Hence, a point will be reached in this process where the stiffness of the superincumbent strata in the centre of the panel becomes less than the post-peak stiffness of the pillars. At that point, the deforming pillars will become unstable and sudden collapse will occur.

Since the stiffness of the superincumbent strata is a function of its modulus, thickness and span, while the post-peak stiffness of a coal pillar is a function of its width-to-height ratio, (see Sect. 4.3.1 and Fig. 4.10), it follows that basic design controls against the onset of sudden, uncontrolled pillar system failure are:

- most fundamentally, panel pillars of sufficient size to prevent pillar working stress exceeding pillar strength;
- panel pillars of sufficient width-to-height ratio to result in their post-failure stiffness being adequate to control the rate of roof convergence if the pillars do fail; and
- interpanel pillars to restrict panel span so that the residual stiffness of the overburden strata is sufficient to control the rate of roof convergence.

It is most important to be aware that:

- There may be few, if any, warning signs of impending failure of overloaded coal pillars. Figure 4.30 shows the state of coal pillars in similar circumstances to pillars in an adjacent active mining panel that failed suddenly and without warning, generating a windblast. The pillars were nominally 10 m square and 6–10 m high on 16 m centres. The probability of failure, which was not appreciated at the time, was 70 % based on the UNSW pillar design methodology. The lack of warning signs is apparent by the appearance of the pillars, in particular, the sharp pillar corners, the visible cutter pick marks and the lack of fretting and spalling (as evidenced by the presence of stonedust).
- Pillar system failure associated with foundation failure may also occur in an uncontrolled manner.
- The stability of a bord and pillar layout is not governed by the behaviour of one or two individual pillars but by the behaviour of all pillars in the layout. In some situations, this includes the interpanel pillars.
- Not all sudden pillar failures are uncontrolled failures. Rather, failure may develop in a controlled manner over a period of time, ultimately reaching a point where some pillars totally disintegrate and trigger a complete collapse of

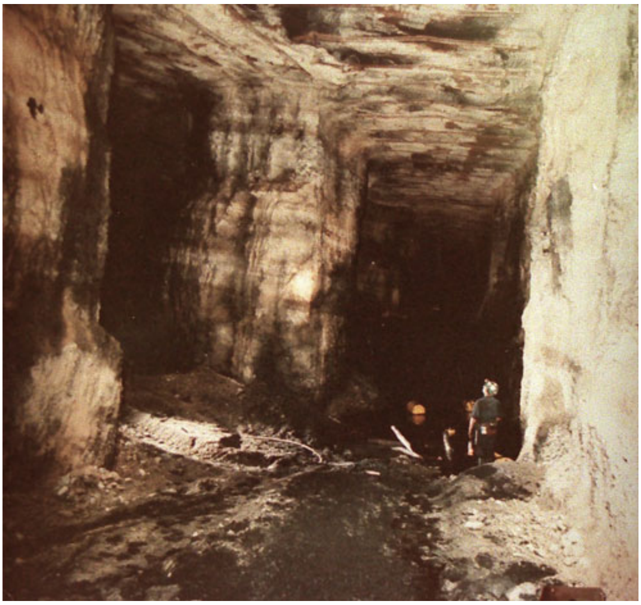


**Fig. 4.30** The state of coal pillars in similar circumstances to an adjacent panel of pillars that failed suddenly and without warning. Note person in bottom right hand corner

the workings. These types of failures are sometimes referred to as a **massive failures** and may include controlled and uncontrolled events.

While there is a lack of information concerning the collapse mode of all failure events in the South African and Australian pillar failure databases, it appears that uncontrolled events have been confined to pillars having a width-to-height ratio of three or less in these countries. Similar observations have been made in the USA (Brown 2001). This suggests that the post-failure stiffness of pillars with a width-to-height ratio greater than three is sufficiently high to control the rate of load shedding and permit the pillars to yield in a stable manner. Figure 4.31 shows a pillar with a width-to-height ratio close to three in the process of failing in a controlled manner.

It is known from instrumentation that fracturing can extend well into the core of pillars with a width-to-height ratio of at least six. The failure of such large pillars requires high loads, generated by mining very wide bords or by abutment stress associated with caving. In these situations, roof instability and pressure bursts are added risks that need careful consideration.

### 4.6.3 Dynamic Confined Core Failure

The dynamic confined core failure mode is related to the lateral extent and severity of ribside deformation. Salamon (1995b) deduced from analytical modelling that circumstances could arise where four distinct zones of behaviour developed within a pillar, namely ‘elastic’; ‘yielding’; ‘crushed’; and ‘slumped’ as shown in Fig. 4.22. Yielding coal softens as it is subjected to strain, causing a reduction in its cohesion. If it is unconfined, it disintegrates when its cohesion is lost and becomes slumped coal. If confined, it eventually becomes crushed coal. These states were investigated in two-dimensions by progressively loading a solid abutment and a long strip pillar by increasing the span of the flanking excavation(s).

In the case of a solid abutment, the analysis revealed that a situation could arise where two zones (elastic and yielding) could be transformed suddenly into four zones, with the potential for this to be violent and to result in a pressure burst. It was found that, depending on their width, similar behaviour could be associated with pillars. Salamon

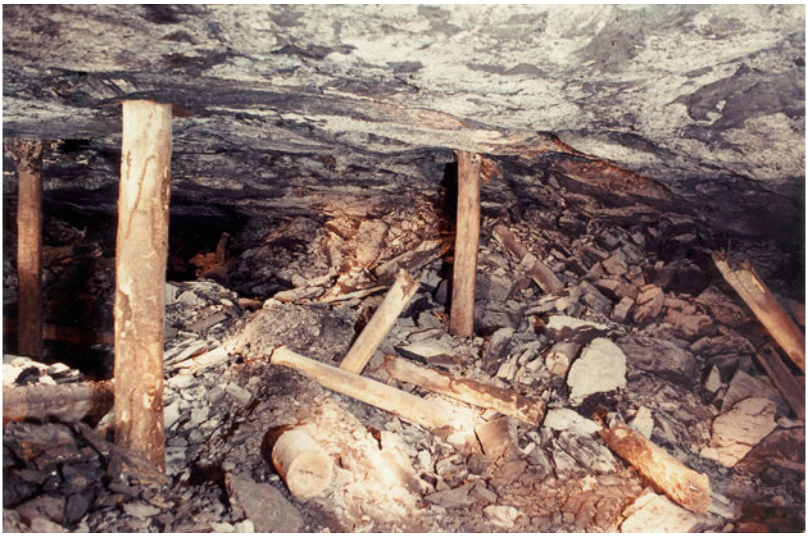


**Fig. 4.31** A pillar that is in the process of failing in a controlled manner

presented his finding under the following three pillar width-to-height ratio categories, stressing that these were mere indications at that time:

- Narrow pillars: w/h <3
- Intermediate pillars: – 3 < w/h <5–7 (?)
- Wide pillars: – w/h >7 (?)

In the case of narrow pillars, the two yield zones coalesce before the vertical stress is reduced to zero at the rib. From that point onwards, the whole pillar is yielding and failure occurs in a 'conventional' manner. The mechanistic basis for defining a pillar to be intermediate was if the width of its elastic core was small when the vertical stress at its edges reduced to zero. Therefore, when spalling of the sides of the pillar starts, there is insufficient width available to enable the development of stable slumped, crushed and yield zones. Salamon concluded that in this case, once the pillar sides are destressed, the failure of the whole pillar is unavoidable and rapid, although the imminence of failure will be visible. Wide pillars were susceptible to sidewall failure and pressure bursts but would not fail as a whole due solely to these behaviours. Salamon (1995b) concluded that *'the research so far has revealed an unexpectedly complex pillar behaviour'*.

A number of events, most notably the Crandall Canyon Mine pillar failure in 2007, appear to give weight to the outcomes of research to date in this area. This incident was associated with a relatively unique set of circumstances. It occurred in the process of extracting panel pillars and barrier pillar coal in a remnant area between flanking longwall panels at a depth range of around 450–680 m. It is likely that these circumstances resulted in:

- pillars also having to bear abutment load from adjacent total extraction workings;
- very high pillar loads;
- the extent of mining in the region being so large that the stiffness of the overburden was reduced significantly, possibly to the extent that the pillars were subjected to full dead-weight load; and
- the load acting on the pillars being sustained as the pillars yielded.

Gates et al. (2008) reported that investigations revealed panel pillars with a width-to-height ratio of almost 8 had failed within seconds over a distance of 800 m and that the barrier pillars to the north and south also failed. The barrier pillar to the south had a width-to-height ratio of about 6.2 at the point where failure was initiated, increasing to 15.4 further outbye. The outbye barrier pillars displayed severe signs of damage but the extent of internal fracturing is unknown.

The day after the initial pillar failure event, a pressure burst occurred in No. 3 entry/roadway (of four) that had been cleared of rubble in an attempt to reach the missing miners. The investigation report describes the entry (at No. 120 - crosscut/cut-through) as having been *refilled with rubble* as shown in Fig. 4.32. A continuous miner was then used to clear No. 1 entry abutting the barrier pillar, with the investigation report stating that this machine *was loading from a rubble pile that resembled an unmined coal face*. During this process and ten days after the first event, a pressure burst occurred at the re-mining face (inbye No. 126 crosscut), resulting in the deaths of another three persons.

It appears that this incident provided a rare opportunity to generate a very high, deadweight load on large width-to-height ratio pillars to test their strength. Based on the UNSW power formula, and disregarding additional abutment loading associated with adjacent total extraction workings, the safety factor of the failed pillars ranged from around 0.8 to 1.3. The extent of fracturing within the pillars is unknown, as is the manner in which the failure process may have been modified by the bords choking off and providing confinement to the pillars.

The incident has caused re-evaluation of the concept that pillars of large width-to-height ratio are indestructible and highlights the incomplete knowledge base regarding pillar mechanics. It reinforces the advice of Salamon (1995b) and NIOSH (2010) that further research is required to understand the failure process associated with squat pillars.

## 4.7 The Complexity of Pillar Behaviour

The variety of approaches to pillar design reflect the diverse functions of coal pillars and their loading conditions and the deceptively complex nature of pillar mechanics. Some fundamental aspects of pillar behaviour, such as the confined core concept, the mechanistic definition of pillar strength, and pillar foundation behaviour (see Sect. 4.8.3) are still not fully understood and require further fundamental research. For example, based on mechanics, the horizontal component of pillar load increases with increase in span of the surrounding excavations, thus increasing pillar confinement, while the vertical component increases with depth, thereby increasing the shear strength of interfaces. Both behaviours could be expected to increase pillar strength

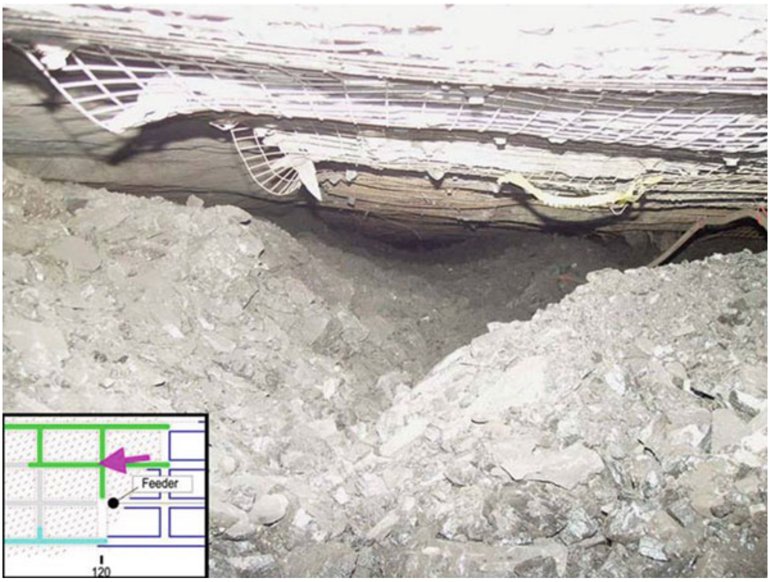


**Fig. 4.32** View of No. 3 entry after 7 August 2007 burst at Crandall Canyon Mine (After Gates et al. 2008)

and, in fact, may have some precedent in practice. For example, Babcock (1969) concluded that the results of a laboratory testing study of the different specimen end constraints suggested that large horizontal stress in orebodies mined by the room-and-pillar method should increase the strength of the pillars. Salamon (1999) questioned if the decrease in coal pillar strength predicted by power law formulae when a pillar of fixed width-to-height ratio is made larger, is actually a size effect or an effect introduced through other field variables, such as bord width not being factored into the formulations.

History indicates that the concept of defining pillar strength on the assumption that both load and strength are uniform across the pillar has considerable merit in pillar design when the in seam coal element is the weakest component in the pillar system. Nevertheless, the concept and the various design formulations based on it are not mechanistically rigorous and comprehensive. This does not appear to be fully appreciated by all, leading to propositions regarding pillar behaviour and design that are tenuous and, sometimes, concerning from a risk management perspective. These propositions tend to be based on pillar width-to-height ratio criteria and concluded from empirical databases and laboratory testing outcomes. It is important to appreciate the uncertainties and limitations associated with some of the factors that underpin these types of approaches.

Most fundamental is that a purely width-to-height ratio approach to stability assessment is limited by the fact that it only considers one component of the pillar system. Furthermore, as reported by Salamon (1992c) in regard to empirically derived pillar strength formulae founded on width-to-height ratio:

> *True to the nature of empirical relationships, these formulae do not reveal anything with regard to the reason for the stated influence of the width-to-height ratio. Thus, they cannot be the basis of an in-depth discussion of pillar mechanics.*

Notwithstanding these aspects, pillar width-to-height ratio has been and continues to be a primary focal point when discussing pillar performance and, in particular, what constitutes an 'indestructible' coal pillar. Holland (1964) reported that if laboratory values for coal strength that he had previously determined (Holland 1942) could be extrapolated to coal pillars in a mine, then pillars having a width-to-height ratio greater than 10 or 12 probably do not fail under any load that can come on them. Wagner (1974), Salamon and Oravecz (1976) and Cook and Hood (1978) all proposed that a coal pillar may be able to support an almost unlimited load once its width-to-height ratio exceeded about 10.

Since around 2000, some researchers have advocated that, from a load carrying perspective, pillars will not fail once their width-to-height ratio exceeds about 6. It appears that this view is partially premised on the size of pillars associated with the South African, United States and Australian databases that support the Salamon and Munro, Bieniawski and UNSW pillar strength formulations. However, the UNSW research also identified a range of other unstable pillars that could not be included in the database because peak pillar load could not be determined to the required level of confidence. This included a number of unstable pillars with a width-to-height ratio in excess of 6.

The exclusion of larger pillars is not surprising since they are not often associated with extensive workings that permit the application of tributary area load theory. Practical mining considerations typically limit the width of bord and pillar panels to less than 200 m and so it follows that tributary loading situations will be mostly confined to shallow depth circumstances, where pillars are concentrated at the lower end of the width-to-height ratio spectrum. Zipf (2001) reported a not dissimilar situation in the USA, noting that although coal pillar failure is more prevalent in mines less than 100 m deep, this may be only a reflection of the prevalence of shallow bord and pillar workings.

A number of reasons may account for why there is a limited number of pillar failures reported at higher end width-to-height ratios. These include:

- Typical bord and pillar mining layouts do not generate the high loading regimes required to cause failure of such large pillars.
- The total collapse of large width-to-height ratio pillars is prevented in conventional bord and pillar layouts because failure is arrested at an early stage by the bords becoming choked with failed material.
- Vertical surface subsidence is constrained as a result of failure being arrested by the choking of bords and, therefore, surface evidence of failure over bord and pillar workings may be missed or misinterpreted as elastic convergence.
- Instability of large width-to-height ratio pillars is more likely to be characterised by strain softening or strain hardening rather than total collapse, in which case failure may go unnoticed for two reasons. Firstly, the high pillar loading regimes are usually generated by the partial extraction of the pillars on retreat out of a mining panel, with no safe access then being available back into these areas in order to detect pillar instability. Secondly, the surface expressions of this type of instability may be minor and, once again, be missed or misinterpreted.
- The very high load carrying capacity of pillars of large width-to-height ratio may result in bearing capacity failure of the roof or floor strata before the peak strength of the pillar is reached.

Given these factors, it is unsound to use statistical analysis of existing databases to draw conclusions as to a width-to-height ratio beyond which pillar load cannot exceed the peak pillar load carrying capacity. Rather than dismiss high end width-to-height data points as outliers, added weight may need to be given to them in recognition of their rare occurrence. Disregarding such data points on the basis that they are too many standard deviations away from the mean of the database to be credible may be fraught with risk.

In motivating the proposition that pillar failure is confined to width-to-height ratios of less than about 6, reference has often been made to one specific set of stress-strain curves derived from a suite of laboratory compression tests on cylindrical coal specimens reported by Das (1986). This select set of outcomes, shown in Fig. 4.33a, has formed the basis for calibrating numerical models in a number of studies. Whilst helpful for conceptualising how pillar behaviour changes with increasing width-to-height ratio and for discussing what actually constitutes pillar strength, there are a number of limitations associated with this specific set of data. These have implications for how the data is used in quantitative analysis and the reliability of the analysis outcomes.

These limitations include that the testing results are specific to coal/steel end interfaces; the test measurements have undergone a correction process in an attempt to compensate for the effect on specimen behaviour of indentation of the steel platens; and the set of outcomes is selective, being only one set of six and significantly different to the other five sets of outcomes. The significance of the latter two factors is evident from comparing the modified test outcomes for the Singhpur Middle Seam shown in Fig. 4.33a with unmodified outcomes for the Kenda Seam shown in Fig. 4.33b.

Platen indentation influences confinement provided to the test specimen. Das (1986) attempted to account for platen indentation by normalising all pre-peak strength slopes to the same value as that of a specimen with a width-to-height ratio of 0.5. As noted in Sects. 2.6.2 and 4.4.5.3, this width-to-height ratio is sufficiently small that the compressive strength of the specimen is unlikely to be influenced by shear stresses induced at the specimen/platen interfaces; a situation that does not apply to larger laboratory specimens and to in situ pillars. The implications of the normalisation approach adopted by Das need to be carefully considered, especially if applying the laboratory test outcomes to the calibration of pillar design procedures.

The use of laboratory testing outcomes for pillar design becomes more problematic when the effects of surface area and volume on laboratory outcomes are considered. By way of

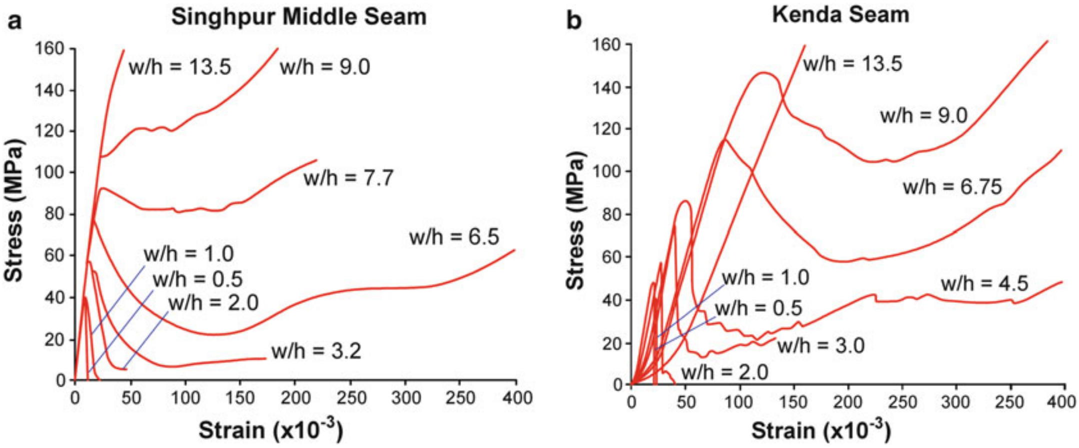


**Fig. 4.33** Influence of width-to-height ratio, w/h, on the strength and post peak strength behaviour of different coals as determined by laboratory testing. (**a**) Modified test outcomes for Singhpur Middle Seam coal samples, (**b**) Unmodified test outcomes for Kenda Seam coal samples (After Das 1986)

example, Das varied width-to-height ratio by varying the height of 54 mm diameter specimens while Madden (1990) produced the outcomes shown in Fig. 2.19 by varying the height of 24 mm diameter specimens. This could also have been achieved by maintaining a constant specimen height and varying specimen width. In that case, if specimen height was fixed at 54 mm, the area of the specimens in contact with the platens would range from 0.25 to 182 times the area associated with the same width-to-height specimens tested by Das (1986). Hence, while the trend of specimen strength variation with width-to-height ratio may have been similar, significantly different stress-strain curves would have been obtained if specimen width rather than specimen height had been varied. A range of other limitations associated with laboratory testing that have implications for using laboratory determined stress-strain curves to calibrate pillar design procedures and to validate numerical models are discussed in Sect. 2.6.2.

The notion that a coal pillar is indestructible once its width-to-height ratio exceeds about 10 appears to have been qualified by many of its proponents. Reference, for example: Holland (1964) – ‘*probably do not fail*’; Wagner (1974) – ‘*are unlikely to fail except by punching into the roof or floor*’; Salamon and Oravecz (1976) – ‘*it can support an almost unlimited load*’ and ‘*there is some evidence that when pillars have a width-to-height ratio exceeding, say 10, they do not fail under any practically possible load*’; and Cook and Hood (1978) – ‘*pillars were thought to be almost indestructible*’. At around the same time, some of these authors made corresponding statements in regard to stabilising pillars employed in tabular metalliferous deposits in South Africa to ameliorate the risk of pressure bursts (rockbursts). These pillars act as interpanel pillars to support bridging strata, analogous to panel and pillar mining in underground coal mines. The main distinctions between the two mining situations are that in the case of metalliferous mining, mining height is typically only 1.2–1.5 m; the compressive strength of the material comprising the pillars is at least an order of magnitude greater than that comprising coal pillars; and the depth of mining can exceed more than 3000 m.

Cook et al. (1977) advised that stabilising pillars should have a width-to-height ratio exceeding 15–20 ‘*to ensure that such pillars are themselves stable*’. Salamon and Wagner (1979) stated that ‘*Experience suggests that beyond a critical width-to-height ratio which is of the order of fifteen to one pillars become* *virtually indestructible. However, consideration must be given also to the effects of high pillar stress on the hanging and foot-wall strata'*. Soon after, Salamon (1980) advised that '*in designing partial extraction layouts, care must be exercised that the width-to-height ratio of the pillars is at least 20. This is required to ensure that these control pillars do not fail due to the excessive load which will act on them.'*

Hence, despite the significantly greater material strength of metalliferous pillars, the width-to-height ratio to prevent failure of the pillars themselves was thought to be 50–100 % greater than that for coal pillars. The explanation may be related to the different role that pillars were envisaged to perform at the time and to the significantly greater loads that the pillars were required to withstand. Subsequently, a number of bord and pillar mining methods have evolved which rely on coal pillars performing in a similar manner to stabilising pillars in tabular hard rock mines but for the purpose of controlling surface subsidence.

One of the largest in situ pillar strength tests was conducted by Wang et al. (1976), who gradually reduced the size of an instrumented, 1.8 m high, square pillar with an initial width-to-height ratio of 12. It was concluded from stress and displacement measurements that the pillar exceeded its peak load carrying capacity at a width-to-height ratio of 10.7, with Skelly et al. (1977) reporting an average pillar load of between 20.02 MPa and 22.8 MPa at that stage. This compares to a predicted range of 28.6–36 MPa using Bieniawski-PSU and UNSW formulations. Given subsequent advances in knowledge and instrumentation, the capacity of the vibrating stress meters to have reliably measured stress changes in the Wang et al. (1976) research may now be questionable.

Bieniawski (1992) reported that the then recent studies (unpublished at the time) suggested that wide coal pillars, even with width-to-height ratios of 15 and over, may have failed, depending on the definition of 'failure'. No further information was apparently forthcoming. Salamon (1992a) qualified the phrase *complete collapse* when stating that:

> *It is difficult, if not impossible, to visualise the complete collapse of pillars of large width-to-height ratios (say, in excess of 10). This is so, even in the event of the occurrence of a coal bump or some other violent failure in the roof or floor strata. Failure of such pillars is more likely to manifest itself by pillar edge crushing, which may or may not be accompanied by roof falls or floor heaving.*

In light of the Crandall Canyon Mine incident, the issue of what constitutes an 'indestructible' coal pillar remains unresolved. The mechanics of squat pillar behaviour does not appear to have evolved significantly since the research of Quinteiro (1993) and Salamon (1995b). Very squat pillar situations in coal mining are usually associated with concentrations of a small number of pillars rather than a regular and extensive layout of pillars. Both hard rock and coal mining experience with squat pillars in highly stressed situations at depth suggests that the design process may need to have a greater focus on the behaviour of the roof and floor strata.

The statement of Salamon (1999) continues to have currency in regard to the complexity associated with pillar systems, namely that:

> *Pillar strength should be computed numerically from sound mechanistic modelling. Now we do have such models. However, to estimate pillar strength, we need to simulate the complex behaviour of failing rock. The number of parameters to describe such a system is likely to exceed ten. How do we determine so many parameters reliably? Do we use back-calculation?*

## 4.8 Pillar Design Considerations

### 4.8.1 Empirical Data Regime

A fundamental principle in empirical research is that the application of outcomes should be confined to situations that fall within the range of data utilised to derive the outcomes. If outcomes are applied to situations outside of that range, it must be undertaken judiciously. In the case of pillar design, the maximum width-to-height ratio of failed pillars in the Salamon and Munro database was 3.37. The UNSW database included one failed case at a width-to-height ratio of 8.16, otherwise the width-to-height ratios of the remaining 15 cases did not exceed 5 (Fig. 4.34). Subsequently, failed cases not involving roof or floor failure have been reported for width-to-height ratios of around 4 in South Africa and up to 5 in the USA, except for the Crandall Canyon Mine incident where the width-to-height ratio of failed pillars ranged from 6.2 to as high as 15.4.

Care is also required at the lower end of the width-to-height ratio spectrum because coal mass strength and geological structure have an elevated influence on the structural integrity of small pillars. Madden (1991) advised caution at depths of less than 40 m on the basis that pillars with width-to-height ratios of less than 1.75, areal extraction of more than 75 %, and a width of less than 6.0 m, have been known to collapse at safety factors in excess of 1.6. Based on field research, Galvin et al. (1994) concluded that empirical pillar strength formulae are particularly prone to over-estimate pillar strength in the presence of geological structures when pillars have a width-to-height ratio less than 4. The researchers recommended a lower bound width-to-height ratio of 2 because of the sensitivity of the strength of small pillars to geological structure and to slight variations in pillar dimensions. Esterhuizen (1997) reached a similar conclusion in respect of geological structure.

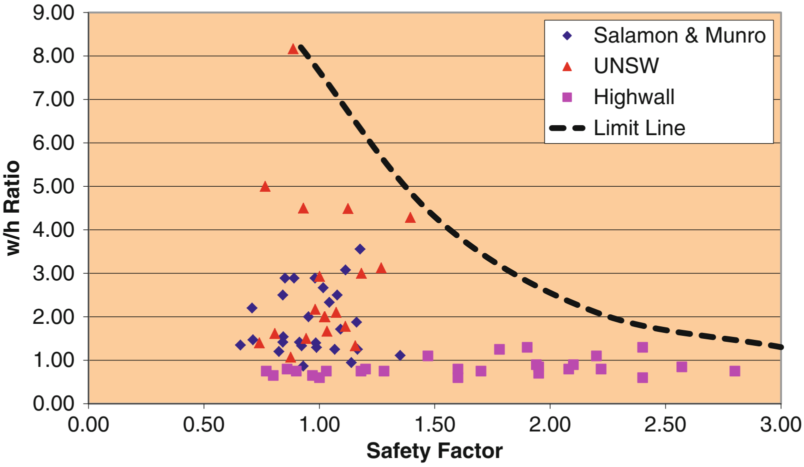
Pillar strength formulae such as those of Salamon and Munro, Bieniawski-PSU and UNSW are not recommended for pillar design in highwall mining situations. Figure 4.34, for example, shows the outcome of applying the UNSW power pillar strength formulae to slender highwall pillars. Clearly, their strength is not predicted by the formulae. The width of slender highwall pillars generally falls outside those in the empirical databases used to derive pillar strength formulae and their width-to-height ratio is generally too small for pillar strength to be influenced meaningfully by confinement. In the absence of geological structure, the strength of slender pillars is governed primarily by material strength.

Geological discontinuities, especially if they are inclined, have the potential to reduce the strength of slender pillars to a negligible value. Numerical modelling by Esterhuizen (2006) of slender limestone pillars, for example, indicated that pillar strength was reduced by as much as 70 % in the presence of an inclined discontinuity. Duncan Fama and Shen (1999) developed a modified form of the Bieniawski-PSU formula to research the stability of highwall mining pillars, concluding that their results implied that due to their narrowness, the stability of highwall pillars had a greater dependence on coal strength than underground coal pillars.

Some design procedures in ground engineering are based on plotting relationships between parameters and enveloping the outcomes within a limit line which may or may not be assigned an equation. An example of this approach in pillar design is shown in Fig. 4.34, where safety factor has been plotted against pillar width-to-height ratio. This specific pillar design approach is not advisable because:

- The relationship between width-to-height ratio and safety factor is not mathematically sensible. Any safety factor can be associated with a given width-to-height ratio value, as evident in Fig. 4.34.
- The lower end of the limit curve is being determined on the basis of applying a pillar strength formulation that is not applicable to very small and slender pillars, such as most highwall pillars.
- The upper end of the limit curve is being determined on the basis of only one data point. There is, as yet, no confirmed mechanistic basis for why a range of safety factors could not be associated with width-to-height ratios at this upper end, just as there is at lower width-to-height ratios. The lone data point may simply reflect a lack of bord and pillar mining layouts with the capacity to generate sufficient tributary load to cause failure of such high width-to-height ratio pillars, and

**Fig. 4.34** An example of an ill-advised w/h – safety factor relationship (defined by the limit line) based on empirical data



the difficulty in detecting failure in these circumstances (see Sect. 4.8.7).

### 4.8.2 Stiff Superincumbent Strata

Despite the fact that theoretical principles point to there being less likelihood of uncontrolled pillar failure when the overburden is stiff, some of the worst mining disasters have been associated with pillar collapse in these circumstances. Three reasons primarily account for these outcomes. Firstly, the stiffer the overburden, the larger the area that has to be extracted before the panel pillars experience deadweight loading, and so the greater the area of mine workings exposed to the consequences of collapse. Secondly, when the effective modulus of the overburden is high, the rate of increase in roof convergence with increase in mining panel span is decreased, thereby often masking signs of impending collapse and/or causing them to be present in old workings that may not be accessible or regularly inspected. Thirdly, the larger the area mined, the higher the likelihood that mining will intersect a fault or dyke, thereby causing a step reduction in overburden stiffness as the bridging beam (plate) is turned into a cantilever.

A particular point to note is that at smaller mining spans, stiff superincumbent strata protects pillars from overburden load, thereby often providing excellent pillar conditions and suggesting higher coal strength than normal. There is a legacy of this condition encouraging either overmining or under-designed pillars. Both approaches result in unsafe mining situations when panel dimensions increase. However, if panel width is strictly controlled and substantial interpanel pillars are left, it can form the basis for increased extraction, possibly utilising yielding coal pillars.

### 4.8.3 Foundation Behaviour

Careful consideration needs to be given to roof and floor strata that have poor strength and deformation properties and, upon exposure, are prone over time to deform under constant load. Their presence considerably increases the level of uncertainty associated with the design of coal pillar systems. Reactive foundation materials that swell upon unloading and exposure to moisture and the mine atmosphere, such as that shown in Fig. 4.35, are also a particular concern. Mills and Edwards (1997) identified a number of cases of excessive surface subsidence associated with pillars with a width-to-height ratio in excess of 12 in the southern region of Lake Macquarie, Australia. Latilla (2004) reported similar outcomes for a number of pillar systems in South Africa that had Salamon and Munro safety factors in the range of 2–3. Similarly, Seedsman (2008) reported that rapid floor failure

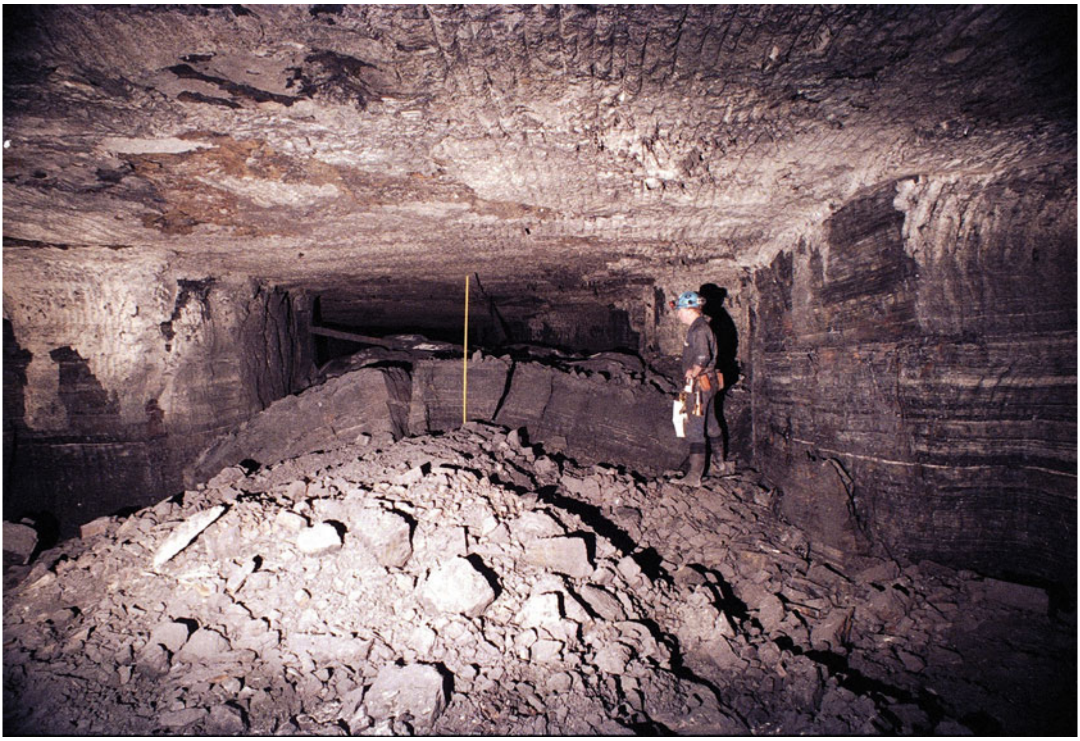


**Fig. 4.35** Floor heave associated with the swelling of a claystone floor with a high montmorillonite content

and subsidence events have occurred in panels that had a safety factor greater than 3 in the Great Northern Seam at Awaba Colliery, in the western region of Lake Macquarie.

Foundation strata that are prone to deform at low load are often loosely and generically referred to in the underground coal sector as 'soft' and/or 'weak'. Typically, they comprise shales, mudstones, claystones, and fireclays. Material properties can range from those of a moderately hard rock through to a saturated soil. Some materials, particularly tuffaceous rock types, have the potential to undergo this full transition over time. In this text, the term **soft** refers to materials that are more soil-like and homogenous with little in the way of defects, so that the low uniaxial compressive strength of these materials is due to the low strength of the intact material. The term **weak** refers to materials that are not necessarily homogenous and have a low strength, typically in a UCS range of 0.5–10 MPa, as a result of the very low strength of the intact material and/or because of a significant density of lower strength defects.

The behaviour of immediate roof and floor strata has implications for the stability and serviceability of excavations as well as for pillar stability. Mining operations tend to avoid soft or weak roof environments because of the additional risk, difficulty and expense in safely managing these conditions, preventing falls of ground and rectifying unstable strata. Hence, research into pillar bearing capacity has focused on floor behaviour. Conceptually, however, the same principles apply to the roof strata.

There have been many attempts to apply civil engineering foundation theory and design procedures to coal pillars. Despite mixed results and the advent of advanced numerical modelling capabilities, this type of approach continues to find application. The Buisman-Terzaghi bearing capacity equation (Terzaghi 1943) for a uniformly loaded, infinitely long strip footing founded on a homogenous incompressible material (Eq. 4.31) forms the basis of a myriad of bearing capacity equations.

$$q_u = cN_c + \gamma dN_q + \frac{\gamma wN_\gamma}{2} \tag{4.31}$$

where

$q_u = ultimate\ bearing\ capacity$
$c = cohesion$
$N_c, N_q, N_\gamma = bearing\ capacity\ factors\ which\ depend\ on\ the\ value\ of\ internal\ friction,\ \phi$
$w = width\ of\ footing$
$\gamma = unit\ weight\ of\ soil$
$d = depth\ of\ footing\ beneath\ the\ surface$

The majority of applications of bearing capacity theory to soft and weak strata in coal mining environments have been based on assuming the worst-case situation that the conditions are undrained, in which case the angle of friction, $\phi$, equals zero. Under normal circumstances, it is difficult to envisage the friction angle of most coal mine strata, including claystone, being less than 10° but there is a lack of shear strength data to confirm this view. Pillar load builds up over a period of time as the mining face is advanced, thereby providing time for pore water pressure to dissipate to some extent and for a partial recovery in friction angle. However, one circumstance that has the potential to cause a severe reduction in friction angle over an extensive area is the rapid loading of coal pillars due to the failure of stiff superincumbent strata.

A number of bearing capacity design outcomes reported as 'successful' have been premised on novel failure mechanisms or on material properties or safety factors that have had to be unrealistically adjusted to replicate field performance. Because pillar foundation behaviour in soft and weak environments can be time dependent, there is a risk that extensive areas of mine workings may become vulnerable to instability with the passage of time, including areas that can no longer be accessed and monitored for signs of developing instability. The implications of flooding of the mine workings at some point in the future is an added factor that may need to be taken into consideration at the design stage.

One bearing capacity formula which has found application in a range of coal mining regions is that of Mandel and Salencon (1969). This formula, given by Eq. 4.32, provides for a thin soft layer overlying an infinitely rigid layer and gives consideration to the ratio of the foundation width to thickness of soft layer, w/t. The influence of w/t ratio on bearing capacity performance is consistent with field experience and analytical considerations. Seedsman (2008, 2012) reported on what appeared at the time to be the successful application of the Mandel and Salencon bearing capacity formula to the design partial extraction pillar systems at Awaba Colliery (Australia), a mine with a history of pillar foundation failure under stiff superincumbent strata. In 2014, surface subsidence in excess of one metre was detected over a portion of these workings.

$$q_u = cN_cF_c = 5.14\ c\ F_c \tag{4.32}$$

*where* $F_c$ *is a function of foundation width to soft layer thickness, w/t, and is given by:*

| w/t | $F_c$ |
|---|---|
| ≤1.41 | 1 |
| 2 | 1.02 |
| 3 | 1.11 |
| 4 | 1.21 |
| 5 | 1.30 |
| 6 | 1.4 |
| 8 | 1.59 |
| 10 | 1.78 |
| Thereafter: $N_cF_c \cong (\pi + 1 + 0.5\ w/t)$ | |

The variety of assumptions and modifications associated with applying classical bearing capacity theory to underground coal environments and the range and accuracy of outcomes give rise to considerable uncertainty about the reliability of this approach. Most formulae are premised on laboratory scale testing, empirical models, and elastic and plastic theory concerned with the behaviour of soils, sands and clays assumed to be homogeneous to a depth of two to three times the width of the footing. Similar conditions rarely exist in underground coal mining environments. Therefore, application of bearing capacity formulations to foundation design in underground coal mining warrants careful risk assessment, with consideration being given, in particular but not exclusively, to the following factors:

- The validity and accuracy of the formulae. There is a range of formulae and a variety of empirically derived bearing capacity factors. A range of values for shape factors has also been proposed in an attempt to adapt the two-dimensional solutions to three-dimensional circumstances. Laboratory scale studies involving sand and clay-like materials have featured strongly in the derivation of these factors and in formulae which address two layer situations. Each of these factors is a source of error in its own right. An arguably greater source of error arises when attempting to apply bearing capacity formulae to a mine environment, because the formulae are tailored to reasonably homogenous foundation materials and do not take account of variable material properties and defects, such as fractures and joints, inherent in coal pillar foundations.
- Material properties. It is well established in civil engineering practice that settlement and bearing capacity calculations are quite sensitive to the accuracy of input data and the reliable determination of the required material properties. There are serious practical, technological and financial limitations to sourcing appropriate, adequate and reliable data from underground coal mine environments, especially prior to mining having taken place.
- Dimensional scale. The width of a pillar footing can be an order of magnitude or more greater than typical civil engineering footings and, therefore, the zone of influence of the footing extends for a greater distance into the foundation. Consideration has to be given to whether the material within this zone of influence is homogenous and, therefore, whether it is valid to apply bearing capacity formulations in the given circumstances.
- Multiple layered situations. The wider footing and higher loading situations encountered in mining environments extend the zone of influence of the footing, thereby increasing the likelihood that foundation response will be affected by the behaviour of multiple layers of stratum. It is likely that these multiple layers will contain a variety of materials, some with contrasting mechanical properties. Classical bearing capacity formulae do not explicitly account for these situations, although some formulae may find application to specific circumstances. Implicit approaches, such as calculating effective material properties of a number of layers and inputting these values into classical bearing capacity formulae, can fail to give proper consideration to specific layers that modify or control behaviour.
- Load scale. The loads to which pillar foundations are subjected are typically at least one order of magnitude greater than those encountered in civil engineering.
- Interaction between footings. The footing width to spacing ratio in a mining environment is inverse to that in typical civil engineering environments, thereby giving rise to a much greater potential for interaction between footings.
- Footing construction. Pillar footings comprise natural, non-reinforced material that is embedded with vertical and horizontal defects (joints, cleats, bedding planes) and is of minimal tensile strength. Conversely, civil engineering footings are usually constructed of quality controlled, reinforced materials of higher tensile and compressive strength.
- Time. Rock mass properties can change over time in a mine environment, especially upon being exposed to moisture or sustained load.
- Flooding. Flooding of mine workings in time to come can have implications for, firstly, the selection of an appropriate bearing capacity formula at the design stage and, secondly, the range of material properties that the design procedure has to cater for over the required period of pillar system stability.

Unfortunately, it is a matter of fact that many of these factors cannot be adequately quantified, irrespective of which approach is adopted to foundation design. Given that classical bearing capacity design approaches continue to find application in some underground coal operations, a more detailed overview of civil engineering foundation theory, bearing capacity formulations and performance is presented in Appendix 4.

Some practitioners have assessed pillar stability in soft and weak foundation environments on the basis of pillar safety factor. Considerable care is also required with this approach since it has a very tenuous relationship with the mechanics of foundation performance and outcomes are site specific. Mills and Gale (1993), Galvin (1996) and Mills and Edwards (1997) variously report on achieving reasonably good correlations between pillar safety factor and pillar system performance in tuffaceous claystone floor environments in the Great Northern Seam in the southern region of Lake Macquarie, Australia. Although the research was based on a variety of pillar strength formulations, findings can be approximated to a common point of reference, chosen to be a UNSW power safety factor in this instance. This shows the various research findings to be in reasonably close agreement, indicating that a UNSW power design safety factor of at least 1.7 is required in that environment to provide for pillar stability in the medium term (life of mine).

From a mechanistic perspective, there is little reason for pillar width-to-height ratio to correlate directly with foundation stability. However, it could correlate indirectly if material properties, lithologies, mining height and depth remain reasonably constant over the area of interest. In these circumstances, variation in safety factor reflects variation in pillar width and, therefore, average pillar bearing pressure and the ratio of pillar width to thickness of soft and weak floor foundation strata.

Numerical modelling can provide valuable insight into potential foundation behaviour modes and likely foundation bearing capacity in a given mine setting. It is particularly useful for undertaking parametric and sensitivity analyses to better understand the nature and level of uncertainty, or residual risk, associated with design procedures. The current state of knowledge, albeit incomplete, can also be exploited to manage risk by undertaking comparative risk assessment. By way of example, Fig. 4.36 shows two mine layouts that have the same average pillar stress and percentage extraction. One is a bord and pillar layout and the other a partial pillar extraction layout, with the latter having been applied successfully to restricting surface subsidence in soft and weak floor environments. The three parameters of primary interest in this case are pillar safety factor, because it correlates in some way with probability of pillar stability; pillar width; and the ratio of pillar width to foundation layer thickness because, according to classical bearing capacity theory, foundation stability increases as both these values increase.

Table 4.5 shows that the partial extraction layout results in more than a doubling of the pillar safety factor, thereby increasing the probability of pillar stability by several orders of magnitude. At the same time, it delivers a 250 % increase in pillar width and, therefore, in the ratio of pillar width to foundation layer thickness, both of which theoretically increase foundation bearing capacity. Hence, subject to the nature of other risks associated with each of these options, the partial extraction layout presents a comparatively lower risk of both pillar instability and foundation instability.

In summary, all pillar system designs must at least satisfy two basic requirements, namely, that the in-seam elements have an adequate safety factor against failure and pillar stress is less than the ultimate foundation bearing capacity. In soft and weak foundation environments, ultimate foundation bearing capacity is likely to be the more critical determinant of the maximum acceptable pillar working stress. However, due to a lack of knowledge regarding foundation behaviour and failure mechanisms in underground coal mine settings, compounded by practical, technological and financial constraints in acquiring appropriate and reliable design data, considerable uncertainty is associated with determining foundation bearing capacity.

Risk management considerations dictate that more research supported by successful field experiences is required to give direction and confidence to applying a civil engineering bearing capacity approach to coal pillar foundation design. If a safety factor approach to pillar system design is adopted in soft and weak

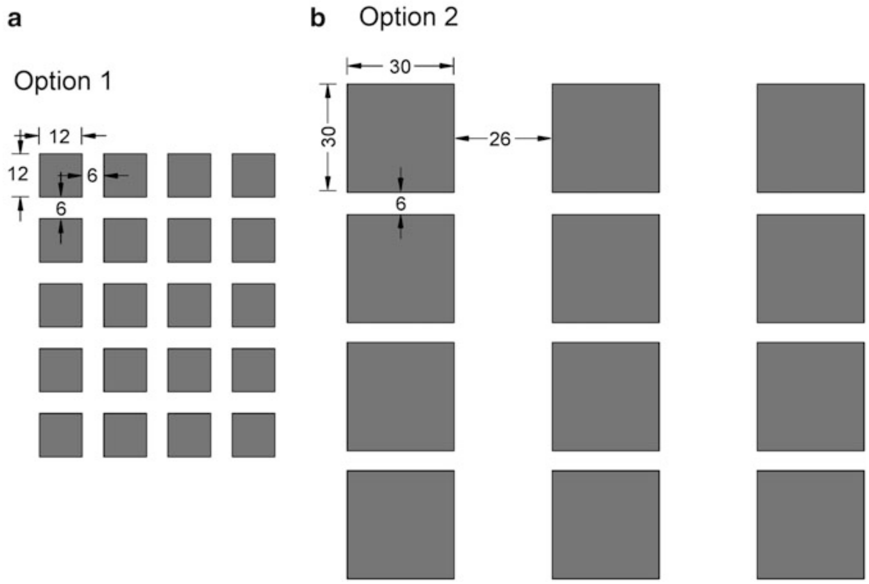


**Fig. 4.36** Plan layout and dimensions of two panel layouts that produce the same percentage areal extraction but which result in significantly different risk profiles for pillar system failure

**Table 4.5** Comparative risk assessment in respect of pillar and foundation stability for two mine design options shown in Fig. 4.36

| | | Option 1 | Option 2 |
|---|---|---|---|
| Depth | H (m) | 100 | 100 |
| Mining height | h (m) | 3 | 3 |
| Pillar dimensions | $w_1$ (m) | 12 | 30 |
| | $w_2$ (m) | 12 | 30 |
| | $\theta$ (°) | 90 | 90 |
| Excavation width | $b_1$ (m) | 6 | 6 |
| | $b_2$ (m) | 6 | 26 |
| Areal extraction | e (%) | 55.6 % | 55.4 % |
| Average pillar stress = Average foundation stress | $\sigma_{aps}$ (MPa) | 5.63 | 5.61 |
| Minimum Pillar Width/Foundation Layer Thickness | $w_{min}/t$ | 12/t | $30/t = 2.5 \times (12/t)$ |
| UNSW Power Safety Factor | | 2.16 | 4.86 |

foundations, it should be calibrated to local conditions. Parametric and sensitivity analysis and comparative risk assessment offer the potential to better quantify design risk, but residual risk is likely to remain elevated until there is a substantial advancement in the knowledge base. Therefore, Ground Control Management Plans are important for managing risk associated with coal pillar foundation instability. These plans should make provision for monitoring the effectiveness of pillar foundation designs and for the periodic review of the state of foundation design knowledge and its implications for the stability of existing mine workings. All design and risk management processes should have regard to the implications on foundation and pillar stability of flooding of mine workings.

### 4.8.4 Seam Specific Strength

The development of pillar strength formulae from back-analysis has been restrained by the lack of databases containing a substantial number of failed cases. Salamon and Munro (1967) and Salamon et al. (1996) addressed this by assuming a uniform intact coal strength across all coalfields in South Africa and in Australia, respectively. The pillar performance experience base that exists in South Africa is now sufficiently large to permit an assessment of the strength properties of specific coal seams in that country. The database has been divided into groups based on both laboratory testing to identify coal seams of similar material properties and on geological and geographical characteristics. Back-analysis of these groupings has produced a number of seam specific pillar strength formulae, each of which can be assigned its own statistical confidence distribution (Salamon et al. 2006).

In some cases, this approach has resulted in a higher standard deviation and, therefore, a lower level of confidence being associated with safety factors. Nevertheless, pillars designed to a higher safety factor, in order to retain the same level of confidence in the design outcomes as in the past, can have smaller dimensions because the seam specific formulae predict a higher pillar strength. This illustrates the advantage of using a risk-based, probabilistic approach to pillar design rather than a pure safety factor approach.

Differences in seam strength are due to differences in the physical, chemical and structural properties of the rock mass. For example, coal pillars in one particular seam in the Vaal Basin in South Africa are prone to fail after a relatively short time even when designed to unusually high safety factors. Underground observations have identified that this behaviour is associated with rapid weathering of the pillars due to their clay content.

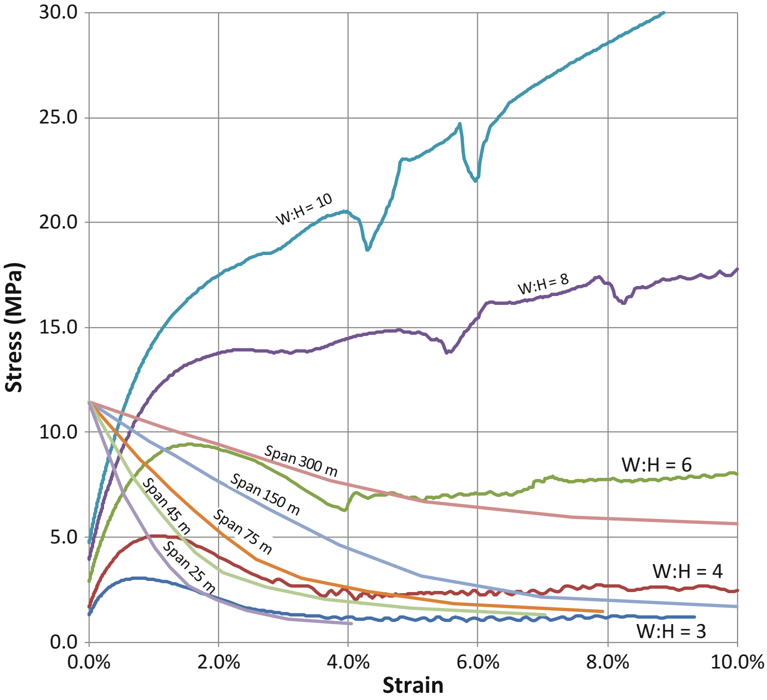
Similarly, a series of coal pillar failures in the Klip River Coalfield in South Africa have been associated with a seam that is heavily impacted by major joint planes (slips). Esterhuizen (2000) utilised numerical modelling to develop equations to explicitly account for the impact of this jointing on coal pillar strength. It was found that the jointing reduced pillar strength by between 41 and 53 %, meaning that a design safety factor of 2.0 effectively equated to a safety factor of around 1. Esterhuizen concluded that when designing pillars, it is not sufficient to account for jointing by simply reducing the rock mass strength. The orientation of joints relative to the direction of loading as well as the width-to-height ratio of the pillars should also be considered.

### 4.8.5 Ground Response Curve

A ground response curve provides a useful means for conceptualising the interdependence between the deformation of a system of coal pillars and deformation of the overburden, as determined by their respective stiffnesses. For all other parameters remaining constant, pillar width-to-height ratio is a measure of pillar stiffness and span is a measure of overburden stiffness. Esterhuizen et al. (2010) adopted the Bieniawski-PSU strength formulation (Eq. 4.29) and utilised numerical modelling to generate stress-strain curves over a range of width-to-height ratios for pillars located in competent roof and floor conditions at a depth of 450 m. Numerical modelling was also used to generate the ground response curve at extraction spans ranging from 45 to 300 m for a loading system comprising strong overburden. The modelling outcomes are presented in Fig. 4.37.

Convergence of the overburden is halted for a given span when the loading line for that span is intersected by a pillar stress-strain curve. The intersection point defines the stress and strain generated in the pillar at that stage. Because pillar stiffness increases with width-to-height ratio, higher stresses but lower strains are generated in

**Fig. 4.37** Pillar stress-strain curves and ground response curves at mid-span of panels with various widths at 450 m depth under strong overburden (Esterhuizen et al. 2010)



larger width-to-height pillars at the time convergence is halted, demonstrating that stiffer pillars attract more load than softer pillars. The analysis indicates that, for the given conditions, pillars with a width-to-height ratio of 6 in a 300 m wide panel (corresponding to $W_p/H = 0.66$) would already be in a critical state of stability during development.

### 4.8.6 Correlations Between Safety Factor and Performance Probability

The correlations between safety factor and probability associated with the Salamon and Munro and the UNSW pillar strength formulae are specific to these formulae and to the assumptions, approximations and data relied upon in their derivations. If data points are added or removed or approximations are changed, the correlations are likely to change.

These safety factor-probability correlations are sometimes applied to outcomes derived using different pillar strength formulations. In marginal situations, it is also not unknown for stability assessments to be based on the Salamon and Munro or UNSW pillar strength formulae but, in an attempt to reach a target stability threshold, to resort to reducing the value of gravitational acceleration from the approximated value of 10 m/s$^2$ back to 9.81 m/s$^2$ so as to gain a 2 % improvement in safety factor. These types of approaches are not consistent with risk management principles. Theoretically, both these approaches invalidate reliance on Salamon and Munro or UNSW safety factor-probability correlations. Such reliances could have serious ramifications, particularly in marginally stable situations. The difference between the probability of instability associated with the same safety factor in the case of the UNSW linear and UNSW power formulae, shown in Table 4.4, illustrates this point.

### 4.8.7 UNSW Pillar Design Methodology

The UNSW pillar design formulae find extensive application. It is important, therefore, that end users are aware of a number of features and limitations associated with this type of back-analysis approach. The Australian database is relatively small in statistical terms, comprising a set of 19 failed cases and a set of 16 unfailed cases, both of which contain 11 cases involving rectangular pillars. There are no diamond (rhomboidal) shaped pillars in the database, although the procedure provides for determining the strength of this shape pillar.

The full range of width-to-height ratios was 0.9–11.2. However, only one of the failed cases had a width-to-height ratio greater than 5 (this being 8.16). Hence, squat pillar effects influenced the estimation primarily through the unfailed cases.

Only the UNSW power formulae incorporate effective pillar width, $w_{eff}$. The UNSW linear formula is based on the minimum pillar width, which may account partially for the higher variance associated with this formula. When pillar strength is calculated using the linear formula, it is a function of only the width-to-height ratio. However, pillar strength calculated using the power law formulae decreases for a given width-to-height ratio if pillar height and pillar width are increased in fixed proportion. This behaviour has been attributed to the effect of volume on pillar strength. However, as noted in Sect. 4.7, it may also be due to the power formulae having been determined from actual mine layouts where other parameters not accounted for also influence pillar strength. The apparent size effect might be introduced through the effect on pillar strength of these other parameters, such as the span of flanking excavations.

Once the minimum pillar width-to-height ratio exceeds 3, the effective pillar width in the squat power pillar strength formula is based on the concept of hydraulic radius. However, the components for pillar volume, $V^{\alpha}$, and width-to-height ratio, $R_{min}$, within the formula (Eq. 4.24) are still calculated on the basis of the minimum pillar width, $w_{min}$, as shown in Eq. 4.27. This may result in the formula underestimating the strength of squat pillars.

Salamon et al. (1996) undertook a sensitivity analysis which indicated that the employment of both effective pillar width and the squat pillar strength formulae improved the reliability of the pillar strength estimations. Nevertheless, it was concluded that the variability and spread of the field data was not sufficient to clarify unequivocally the role of effective width and that of the squat pillar strength formula. Therefore, in situations where the consequences of failure are high, consideration should be given to basing pillar design on the minimum pillar width, $w_{min}$.

Both the linear and power law strength models are proportional to a ‘k’ factor ($k_1$ or $k_2$). Mathematically, this multiplier is the compressive strength of a cube of coal with edge lengths of 1 m. It has been interpreted in this manner in the past, for example, to ascribe a coal strength value of 7.2 MPa (from Eq. 4.23) to South African coal seams.

Theoretically, this simplistic concept is not robust, albeit that it may have produced values in the past that happen to be good approximations of the strength of a one metre cube of coal. When the ‘k’ factor is allowed to float in the statistical analysis, along with the ‘α’ and ‘β’ (or ‘a’ and ‘b’) factors, the back-analysis assigns values to k, α and β which result in the estimated failure strength most closely matching the predicted failure load. This means that two equations with quite different ‘k’ values can produce very similar strength estimations. The variations in ‘k’ values produced from the statistical analysis of full size pillars merely imply that extrapolation of specimen strength from full size pillars is no less difficult than extrapolation in the other direction (Salamon et al. 1996).

There is little doubt that the ‘k’ factor is a material property. However, it is impractical to determine its value from testing alone. Theoretically, numerical modelling provides an alternative approach but the number of parameters required to reliably simulate the complex

**Table 4.6** Comparison of pillar strength parameters and standard deviations for UNSW formulations with and without w/h = 8.16 data point

| | | Linear law | | | Power law | | | |
|---|---|---|---|---|---|---|---|---|
| Method of solution | Data | $k_1$ (MPa) | r | Standard deviation | $k_2$ (MPa) | α | −β | Standard deviation |
| Max. Likelihood | Salamon et al. (1996) | 5.123 | 0.556 | 0.207 | 8.595 | 0.506 | 0.835 | 0.157 |
| | Re-run published database | 5.161 | 0.559 | 0.205 | 8.473 | 0.508 | 0.826 | 0.157 |
| | Re-run with no w/h = 8.16 data point | 5.239 | 0.584 | 0.205 | 8.460 | 0.485 | 0.792 | 0.158 |

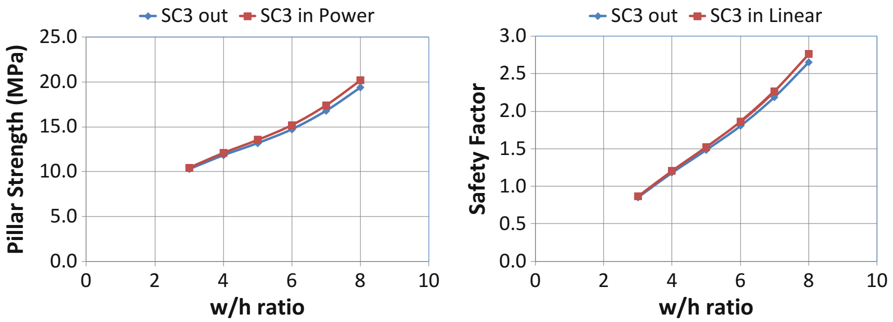


**Fig. 4.38** An example of the effect on predicted pillar strength and safety factor of removing the failed data point of width-to-height ratio = 8.16 from the UNSW database (h = 3 m, b = 5 m, H = 300 m)

behaviour of failing rock for this purpose is likely to exceed 10. The accurate determination of all of these parameters is currently impractical.

Similar to the Salamon and Munro database, the UNSW database was compiled under strict confidentiality agreements with industry. Ultimately, some 30 years later, Salamon reluctantly released information identifying the location of the South African data points, only because an incorrect version of this information had begun to appear in RSA (Salamon 2006). The Australian database was much more difficult to obtain and was either returned or retained under tight control because of the implications it had for the prosecution of mine owners and mine managers, the success of insurance claims, common law claims and company reputation. That underpinning source information which was not retained by or returned to its industry owners was destroyed in accordance with a commitment given to industry (Galvin 2010).

Subsequently, the merits of including the data point with a width-to-height ratio of 8.16 have been queried. Therefore, the analysis has been rerun to test the influence of this data point on the formulations, with outcomes summarised in Table 4.6. In the absence of the original working papers, the Salamon et al. (1996) database was first rerun as a check on the integrity of the analysis. This produced very similar outcomes as shown in Table 4.6, with the differences presumed to be associated with rounding off errors.

The removal of the width-to-height data point of 8.16 has minimal impact on the formulations. This is illustrated for the power law formulae in Fig. 4.38 for the case of 200 m deep workings comprising 3 m high, 5 wide bords. Predicted pillar strength and safety factors change by less than 3 %, with the new power strength formulae being more conservative; that is, they result in a larger pillar size. The correlation between safety factor and probability of stability changes by less than 0.01 %.

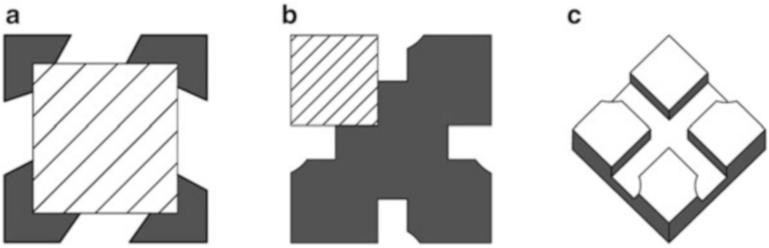


**Fig. 4.39** Three systematic variations to forming irregular shaped pillars in bord and pillar mining. (**a**) Pillar stripping, (**b**) Pillar pocketing, (**c**) Hybrid

### 4.8.8 Diamond Shaped Pillars

Developments in mining technology and ground control strategies are resulting in an increased use of diamond shaped pillars. Caution is always required when dealing with pillars of this shape because of the propensity for the two acute corners of the pillar to be damaged by equipment, to spall on cleat planes or to fail under low levels of vertical stress. These effects can result in a significant reduction in the load carrying area of the pillar while, at the same time, increasing the load acting on the remaining portion of the pillar. Average pillar stress may increase significantly. The impacts on pillar stability are more pronounced for smaller pillars.

Consideration should be given to increasing design safety factors as a control for managing these uncertainties until their effects can be verified and the associated impacts quantified. In all cases, the design of diamond shaped pillars should be premised on site inspections and consideration of the geotechnical environment and be followed up with site monitoring and performance review.

### 4.8.9 Irregular Pillar Shapes

Three variants of bord and pillar mining that result in a systematic layout of irregular shaped coal pillars are **pillar stripping**, **pillar pocketing** and **hybrid pillars**, illustrated in Fig. 4.39. Pillar stripping involves reducing the area of a pillar on retreat by mining a strip off one or more of its sides. Pillar pocketing involves driving stubs into a pillar from one or more of its sides. Both of these techniques are based on achieving the final pillar design safety factor by reducing the area of oversized pillars on the retreat. Hybrid pillars are formed by restricting the mining height in selective bords in multi-slice thick seam operations.

The calculation of pillar load and pillar strength for pillars of the types shown in Fig. 4.39 is clouded in uncertainty. While numerical modelling can be insightful for assessing pillar load, the strength of irregularly shaped pillars largely remains an unknown that has to be estimated. In the case of pillar stripping, the load carrying capacity of the final pillar might be approximated to that of a square pillar of the size shown cross-hatched in Fig. 4.39a. Such an approach has to make some allowance for the increased pillar load associated with the wider effective bord width. Similarly, in the case of pillar pocketing shown in Fig. 4.39b, the load carrying capacity of the final pillar might be approximated to be four times that of the cross-hatched portion of the pillar.

The practical execution of these types of layouts presents additional challenges in that it is easy (and tempting) to drive stubs over-length and/or over-width. A small change in either dimension can make a large difference to the stability of the pillar system, especially if the primary pillar is already relatively small. Strong operating discipline and supervisory control, supported by robust pillar stability analysis, are required when these forms of bord and pillar mining are practiced.

### 4.8.10 Highwall Mining

The strength of pillars utilised in highwall mining is governed by the same principles that apply to larger pillars but, because highwall pillars are very narrow and slender, material strength and geological structure are the dominant determinants of their strength. For example, a structure as simple as a single inclined joint can destroy the structural integrity of a highwall pillar. Because areal percentage extraction is high, substantial load is then transferred to adjacent pillars, thereby increasing the potential for a sudden failure of the pillar system, accompanied by a windblast.

The reader is referred to aspects relating to the strength and design of highwall pillars noted in Sect. 4.8.1. Other ground engineering factors that should be taken into consideration when designing highwall mining layouts include:

- The stability of the highwall prior to commencing highwall mining.
- the effect of undermining on the stability of the highwall.
- The threat to the safety of highwall mining operations presented by the stability of the highwall.
- The impact of previous blasting on the structural integrity of the pillars and surrounding strata.
- The impact of future blasting on the stability of the underground highwall workings.
- The implications on stability of the entries being left in an unsupported state. In particular, roof falls may equate to an increase in effective mining height and, hence, a reduction in pillar strength. Auger mining generally results in more stable roof conditions.
- The likelihood of deviation in drivage (plunge) direction and the implications of this for pillar width and, therefore, stability. A particularly adverse situation arises when drivages intersect (hole into each other) since, not only is the pillar removed, but also the span of the excavation is more than doubled. In addition to increasing the likelihood of roof falls, this can result in the roof falls extending to a greater height, causing a greater reduction in the effective width-to-height ratio of the panel pillars.
- Stress concentrations beneath highwall benches, which may exceed tributary area load. These have been implicated in pillar failures (Duncan Fama and Shen 1999).
- The rapid reduction in loading stiffness as more entries are developed, arising from highwall mining being undertaken at shallow depth under superincumbent strata that is no longer supported on all sides. The combination of very low width-to-height ratio panel pillars and a soft loading system means that unless barrier pillars are left on a regular basis, pillar failure is likely to occur in an uncontrolled manner; that is, sudden and with little or no warning.
- Timing of the sealing of entries. Entries may be sealed soon after formation for a range of reasons associated with safety and the management of water and gas. This material can obstruct observation of pillar behaviour and signs of instability and impending failure.
- The impacts on stability of the flooding of entries.

These considerations reflect the general case with shallow underground mining operations that a small change in material properties, geology or mining geometry can have a large impact on stability. More detailed information on research into highwall pillar design and mine stability is provided in Duncan Fama and Shen (1999), Fama et al. (2001), and Medhurst and Brown (1996, 1998).

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# Interaction Between Workings

# 5

## Abstract

Except in some very shallow situations, mining effects on the rock mass extend laterally beyond the boundaries of mine workings, decaying with distance. In all cases, mining effects also extend vertically into the roof and floor strata and, similarly, decay with distance. Hence, mining panels in the same seam and in different seams interact if they are sufficiently close. Assessment of this interaction requires consideration of the behaviour of excavations and pillar systems and of the properties of the surrounding strata.

This chapter applies the basic physical and mechanical principles of rock behaviour established in preceding chapters to consider firstly, interaction between workings in the same seam and, secondly, interaction between workings in multiseam situations. It has a particular focus on understanding and conceptualising how stress and deformation are distributed about mine workings. This forms the basis for designing mine workings in manners that not only avoid exposure to excessively high stress concentrations but also provide opportunities to exploit stress relief methods on both a local and regional scale.

Issues examined in this chapter include chain pillar design; stress notching in longwall gate-roads; stress notching between mining panels; optimising extraction direction in both single seam and multiseam mining situations; exploitation of sacrificial roadways and goaves to create stress relieved zones; superpositioning of bord and pillar workings; superpositioning of total extraction panels and panel entries in multiseam mining situations; optimising the sequence of extracting multiple seams; the impact of remnant pillars; and the potential for inrushes in multiseam mining situations.

## Keywords

Abutment stress • Barrier pillar • Chain pillar • Domino failure • Double stress notch • Extraction panel • Flexure zone • Gateroad drivage • Horizontal stress • Horizontal stress relief • Inrush • Interaction between workings • Interpanel pillar • Multiseam design • Multiseam interactions •

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Multiseam workings • Panel orientation • Pillar punching • Remnant pillar • Sacrificial roadway • Secondary extraction • Sequence of extraction • Stress notching • Stress shadow • Surface subsidence

## 5.1 Introduction

Assessment of the interaction between mine workings requires consideration of the behaviour of excavations, pillar systems and the surrounding strata. Chapter 3 (Excavation Mechanics) and Chap. 4 (Coal Pillar Systems) present the fundamental principles that govern how these elements behave and underpin the application and extension of the principles to evaluating interaction between mine workings.

Except in some very shallow situations, mining effects on the rock mass extend laterally beyond the boundaries of a panel, decaying with distance. In all cases, mining effects also extend vertically into the roof and floor strata and decay with distance. Hence, mining panels in the same seam and in different seams will interact if they are sufficiently close. There are many permutations of mining method and layout but, basically, these can be classified as follows for the purpose of considering interaction:

- Workings in the same seam, where the interaction is between:
  - First workings (bord and pillar panels)
  - First workings and secondary extraction panels
  - Secondary extraction panels
  - First workings and/or secondary extraction panels in multiple overlapping horizons in the same (thick) seam
- Workings in different seams, where the interaction is between:
  - First workings
  - First workings in some seams and secondary extraction panels in other seams
  - Secondary extraction panels in two or more seams

Additional permutations arise in regard to the order in which seams are extracted, which may be:

- Descending
- Ascending
- Simultaneous
- Random

Since it is not feasible to address all of these permutations in this text, the foundation principles are presented as a basis for the reader to assess their own specific combination of conditions. Thick seam mining techniques that involve multiple mining horizons or multiple slices within the one coal seam are not considered other than to note that the principles pertaining to multiseam mining are also relevant to these situations. Further discussion of the interactive geotechnical and mining considerations associated with thick seam mining systems can be found in many publications including McCowan (1987), Hebblewhite et al. (2002), Bigby et al. (2013), Hebblewhite (2013) and Prusek et al. (2014).

## 5.2 Workings in the Same Seam

### 5.2.1 Framework

Essentially, all underground coal mining methods are comprised of three building blocks, being roadways, pillars, and extraction panels. Different mining methods are distinguished by differences in the dimensions of these various components and the extent of secondary pillar extraction. For example, longwall mining effectively equates to the secondary extraction of a very large pillar surrounded by roadways and smaller pillars.

Each of these components is first considered in its own right as a foundation for assessing a range of combinations of mining method and mining layout within the same seam. The assessment process requires a particular focus on the magnitude and distribution of vertical stress when considering pillar systems and on the magnitude and direction of lateral stress when considering roadways and isolated extraction panels. A focus on both vertical and lateral stress fields is important when assessing interaction between mining panels.

### 5.2.2 Pillar Systems

Fundamentally, the loading within a coal pillar is a reflection of the degree of interaction between two or more excavations. The simplest case to assess is a panel of pillars in which caving does not occur. As shown in Fig. 3.2b, pillar formation in these circumstances results in an increase in horizontal stress in the immediate roof and floor strata as a result of the in situ horizontal stress having to deviate around the roadways. The increased stress has implications for the stability and support of the roadways and might, as discussed in Sect. 4.7 make some contribution to increasing pillar strength. However, unlike vertical stress distributions, horizontal stress distributions in pillar panels are relatively insensitive to the lateral layout and extent of pillar panels and, therefore, are not discussed further in this context. Horizontal stress takes on much greater importance in total extraction settings because of the significant increase in the size of the voids around which the horizontal stresses much deviate. This aspect is considered when discussing roadways and panels.

Figure 5.1 shows how vertical stress is distributed along a horizontal plane through the mid-height of an excavation formed in a primitive stress field where caving does not occur. The floor of the excavation is totally stress relieved, with this stress relief matched by an equivalent increase in vertical stress in the abutments of the excavation, so that the area of zone $A_1$ equals the area of zone $B_1$.

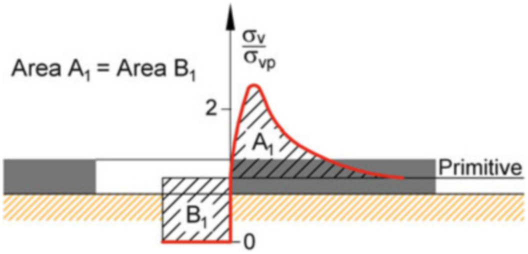


**Fig. 5.1** Conceptualisation of how vertical stress is distributed about an isolated excavation at mid-height in a primitive (virgin) stress field for a non-caving situation

The formation in the seam of a second excavation of the same dimension as the first gives rise to a pillar. Figure 5.2 a shows that if this pillar is sufficiently wide, the abutment stress profile in the freshly formed pillar side will be identical to that of the original opposite abutment and, furthermore, interaction between the two profiles will be negligible. However, as the pillar width is progressively reduced a point is reached where the two original profiles begin to overlap. From that point onwards, the profiles need to be added as shown in Fig. 5.2b to produce the new resultant stress profile within the narrower pillar. Ultimately, with continuing reductions in pillar width, the pillar becomes too narrow for the abutment stress to return to near its primitive value within the pillar (Fig. 5.2c). This results in a change in the shape of the curves that define the contribution of each excavation to stress increase in the pillar. These new curves are still added to produce the overall profile of vertical stress distribution in the pillar which, as reference to Fig. 5.2c shows, is now significantly different to that in the wider pillars.

These concepts are applicable to all types of pillars, including pillars that abut goaf edges. However, the assessment process is rarely applied in practice to panel pillars associated with regular layouts of bord and pillar workings. This is because design and assessment in these circumstances is usually based on the concept of average pillar stress, calculated by dividing tributary area load by pillar area. The effect of that approach has been illustrated in Fig. 5.2 by plotting the average pillar stress for each of the mine geometries used to develop the conceptual abutment stress profiles. In the moderate and high pillar width cases, depicted in Figs. 5.2a, b, respectively, the actual pillar stress is higher than the average pillar stress towards the pillar sides but lower at the core of each pillar. This is not a serious concern in this case because the pillars are sufficiently wide for fractured and failed perimeter coal to still be able to generate significant confinement to the pillar core. However, the situation is reversed in the case of the narrow pillar shown in Fig. 5.2c, where the average pillar stress is now lower than that acting on

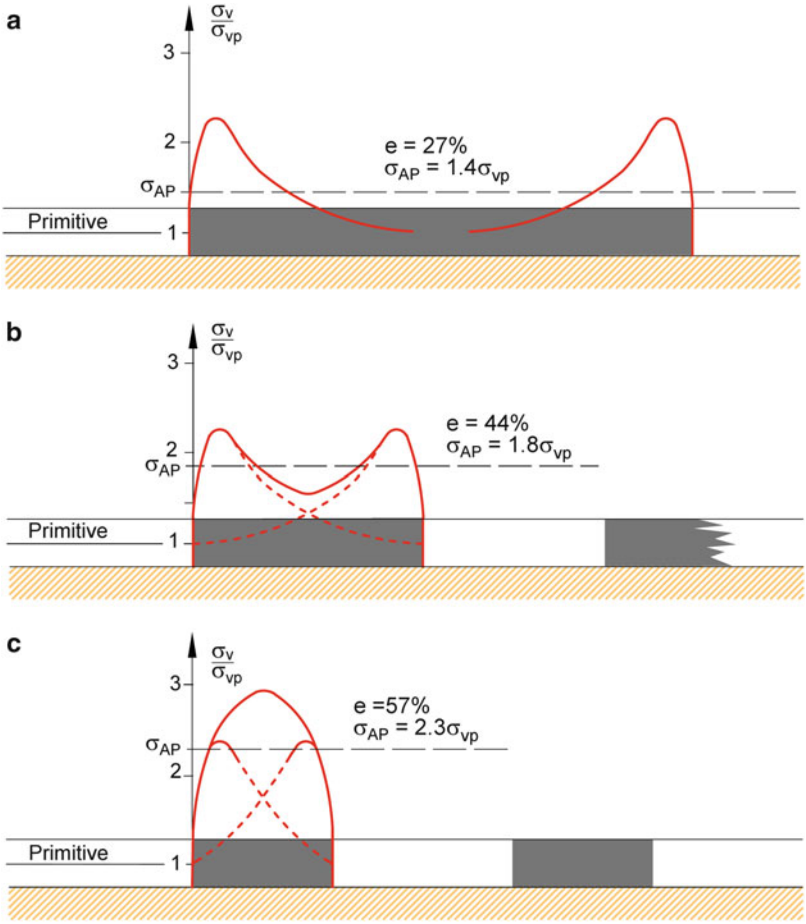

a
$\frac{\sigma_v}{\sigma_{vp}}$
3
2
$\sigma_{AP}$
1
Primitive
e = 27%
$\sigma_{AP} = 1.4\sigma_{vp}$
b
$\frac{\sigma_v}{\sigma_{vp}}$
3
2
$\sigma_{AP}$
1
Primitive
e = 44%
$\sigma_{AP} = 1.8\sigma_{vp}$
c
$\frac{\sigma_v}{\sigma_{vp}}$
3
$\sigma_{AP}$
2
1
Primitive
e =57%
$\sigma_{AP} = 2.3\sigma_{vp}$

**Fig. 5.2** Conceptualisation of how vertical stress magnitude and distribution in a long (strip) pillar are influenced by the distance between the flanking excavations. (**a**) When roadways are sufficiently far apart, associated abutment stress profiles do not overlap. (**b**) As percentage extraction increases and pillar width is reduced, abutment stress profiles begin to overlap and result in an increase in average pillar stress. (**c**) Further increase in percentage extraction and associated decrease in pillar width results in a change in pillar stress profile and elevated stress.

the core of the pillar and, therefore, of more concern.

The situation becomes more complex when some or all of the pillars within a panel are extracted. Insight into these interactions is provided by considering the distribution of maximum principal stresses around an isolated panel in the extreme case of a strong, stiff bed spanning a 200 m wide excavation at a depth of 100 m, depicted in Fig. 5.3, for an assumed horizontal to vertical stress ratio of 0.5. These types of non-caving situations have been associated with conglomerate and sandstone strata and with dolerite sills. Normalising the maximum principal stresses relative to the primitive vertical stress highlights that a zone of high stress concentration exists ahead of and above and below the face (Fig. 5.3). However, the areas above and below the extraction panel are stress relieved and, in some circumstances, may even be subject to extension. This can have important implications for multiseam mining.

The existence of a zone of high stress concentrations ahead of and above and below the secondary extraction abutments is particularly important when secondary extraction is contemplated in an area where bord and pillar mining has already occurred. In the case of first workings in the same seam, it is important that sufficiently large interpanel pillars and barrier

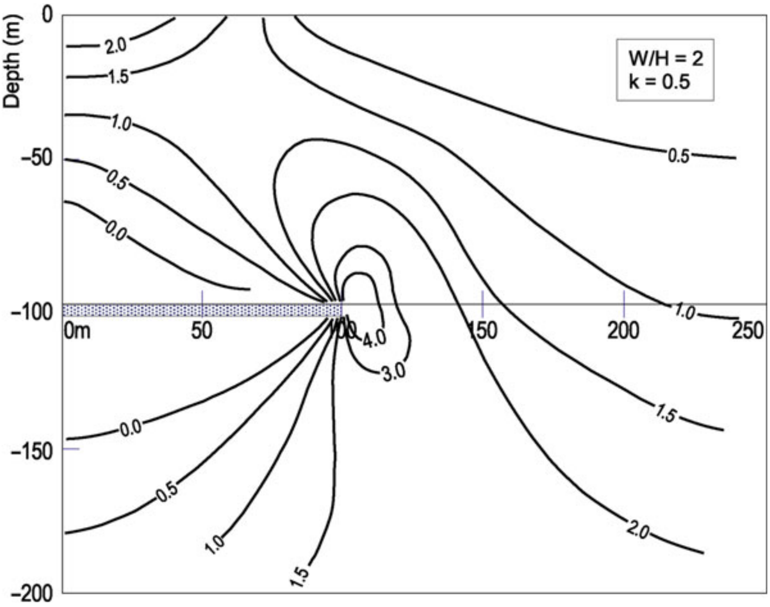


**Fig. 5.3** Normalised principal stresses around a shallow isolated panel that has not caved (Adapted from Galvin et al. 1982)

pillars are left between the total extraction panel and surrounding first workings. The stress contour line labelled ‘2.0’ in Fig. 5.3 indicates that the stresses within the area delineated by this contour are at least twice the value of the primitive vertical stresses. Therefore, bord and pillar workings which were designed to a safety factor of, say, 1.6 would be overloaded and pillar failure would be likely as a result of the abutment stresses associated with the total extraction panel.

Compartmentalisation of bord and pillar and partial pillar extraction workings by leaving substantial pillars between panels on a regular basis mitigates the risk of pillar failure occurring in an uncontrolled manner and aids in controlling the extent to which a collapse, controlled or uncontrolled, may spread throughout the workings. These pillars are referred to as **interpanel pillars** in this text. In the USA, this type of pillar may be referred to as a **barrier pillar.** Interpanel pillars perform a range of functions, depending on the mining method. One of the most important of these is that interpanel pillars enhance the stability and safety of mining panels that are supported by pillars, including partial pillar extraction panels. The interpanel pillars are usually wider than the panel pillars and effectively continuous (that is, they contain few, if any, cut-throughs). In these situations, they function as engineered controls in a number of ways, the most notable being:

- Because of their greater stiffness, interpanel pillars may carry a significant portion of the weight of the overburden above the panel pillars, thereby further enhancing mine stability. This characteristic is deliberately exploited by restricting panel width in some mine layouts, such as those associated with yielding pillars and highwall mining.
- They act as a buffer to protect panel pillars from abutment stress associated with mining operations in adjacent mining panels.
- The increase in the ratio of pillar width to thickness of soft or weak foundation layers can be exploited to improve foundation stability (see Sect. 4.8.3).
- Interpanel pillars have spare load carrying capacity to help compensate for the unexpected, such as roof falls, geological weaknesses, and reduced load bearing capacity of the pillar foundations.
- Should the panel pillar system gradually become unstable, interpanel pillars may retard the rate and extent of failure and limit the underground and surface impacts of the instability.

- In the event that the panel pillar system fails suddenly, interpanel pillars play an essential role in preventing the collapse from spreading rapidly and violently throughout the mine. There is a high likelihood that the travelling abutment stress front generated as a result of failure of panel pillars will be arrested by the interpanel pillars due to their increased load carrying capacity and stiffness, thereby terminating the so called ‘pillar run’ or ‘domino effect’.

In some circumstances, the safety factor of the panel pillars may be so high that the panel pillars might be considered as interpanel or barrier pillars in their own right. Nevertheless, compartmentalization of workings on a regular basis is still advisable. History has repeatedly shown that the apparent stability of panel pillars can give rise to the temptation not to utilise interpanel pillars, or else, to subsequently partially or totally extract them. This is despite recommendations arising out of a number of enquiries into mining disasters, some dating back to the late eighteenth century, to compartmentalise mine workings on a regular basis.

The inquiry into the collapse of Coalbrook Colliery (Moerdyk 1965) reported that *‘mining should be carried out in panels surrounded by barriers of unworked coal of dimensions which will limit subsidence to a single panel in the event of pillar collapse.’* Galvin and Anderson (1986) stressed that: *‘The leaving of interpanel pillars on a regular basis to prevent a pillar collapse within a panel from spreading throughout the mine workings is a fundamental design requirement in the layout of extensive bord and pillar workings.’* Hebblewhite et al. (2002) reported: *It is a matter of great concern that even today we can find mine layouts which are not heeding this recommendation (*of Moerdyk 1965*) and operations mining personnel or their technical advisors who do not appreciate this.* At about the same time, Chase et al. (2002) reported that a key finding of their research into pillar extraction at depth was that substantial barrier pillars were essential to maintain stability when mining depth exceeds 1,000 ft (~330 m).

Some 45 years after the Coalbrook disaster, the inquiry into the Crandall Canyon Mine disaster (NIOSH 2010) concluded that: *‘Had substantial barrier pillars been used to compartmentalize this large area, and to isolate it from the adjacent longwall gob areas, a mine collapse of the magnitude of the one that occurred would have been highly unlikely if not physically impossible. In the future, the consistent application of properly sized barrier pillars between retreat mining panels should go a long way towards reducing the risk of bursts in deep cover room-and-pillar mines.’*

The situation is somewhat different for total extraction workings in the same seam. In these cases, interpanel pillars which, importantly, includes longwall chain pillars, perform a range of additional functions that include:

- providing a buffer of sufficient width between two panels such that the serviceability of the development roadways of a current extraction panel are not unduly impacted by high abutment stress associated with a previously extracted panel;
- controlling surface subsidence by preventing two subcritical panels joining up to become a critical or supercritical panel;
- ventilation management;
- water management; and
- spontaneous combustion management.

Wilson (1972) recognised the importance of roadway serviceability in longwall mining, suggesting that chain pillars must be large enough to handle the abutment loads while also securing the integrity of longwall gateroads. Salamon (1991a) noted that the avoidance of pillar collapse is merely a necessary but not a sufficient condition for ensuring the proper functioning of a roadway entry system.

Maintaining roadways in a serviceable state requires that interpanel design has regard to local geology, abutment stress magnitude and profile, mining height, the type and density of support to be installed in the roadways, and the duration and level of serviceability required of the roadways. In the case of longwall mining, two opposing criteria have to be satisfied. Firstly, it is desirable that the interpanel pillar be as narrow as possible to minimise development drivage and to maximise resource recovery. Secondly, interpanel pillar width should be as wide as possible to protect the tailgate from the effects of high abutment stress (Galvin et al. 1982).

Care is required with empirical approaches to interpanel pillar design in total extraction situations. Nearly all approaches rely on pillar strength formulations derived for non-caving situations and, therefore, for very different load and confinement domains. Many attempts to estimate interpanel pillar load by utilising the concept of an abutment angle which, as discussed in Sect. 3.3, has serious limitations at depths greater than about 200 m because it does not take account of the stiffness of the loading system. At shallower depths, an abutment angle/prescribed abutment load distribution profile approach can be quite useful for approximating abutment load, provided that the superincumbent strata does not bridge. The formulations for calculating abutment load in the most general case for these circumstances are presented in Appendix 5.

Even when pillar load is known accurately, the formulations are still usually based on average pillar stress and, therefore, do not take account of peak stresses or the non-symmetrical distribution of abutment stress across an interpanel pillar. In some instances, pillar strength formulations are applied with the aim of preventing working stress from exceeding pillar strength throughout the full life cycle of the pillars while in others, they are correlated in some manner to the state of roadway roof, floor and sides throughout the full life cycle of the roadway. Some empirical approaches to interpanel pillar design also have a high reliance on parameters such as a rock mass rating that may not have a strong mechanistic connection to system behaviour, while failing to consider some parameters that do, such as floor and roof bearing capacity and, in particular, the overall stiffness of the loading system.

Interpanel pillar design in caving situations is an area that warrants further research. Given the safety implications associated with the loss of ground control and subsequent recovery operations, the capital cost of a longwall installation, operating costs associated with maintaining roadways and chain pillars that are in poor condition, and the lost revenue when ground control is adverse, the question arises as to how can risk associated with interpanel design be reduced. Numerical modelling design procedures provide an alternative approach to interpanel pillar design and have been promoted for this purpose since at least the early 1980s (see, for example, Galvin et al. 1982). However, serious impediments can be associated with these approaches in caving situations, especially in regard to quantifying pillar load, the effect of caving on pillar strength, and goaf reconsolidation characteristics.

Notwithstanding this, the cost of undertaking parametric and sensitivity analyses utilising sensible numerical models to gain insight into and to improve confidence in interpanel pillar design is minor to trivial in comparison to the safety and financial risks and the due diligence obligations of the mine owner. In all cases, the monitoring of field performance and the modification of interpanel widths and associated support procedures on the basis of monitoring outcomes assumes a high importance in managing uncertainty associated with interpanel pillar design in caving environments.

### 5.2.3 Roadways

The vertical stress distribution about a roadway has implications for rib loading and stability. However, these tend to be secondary to those of horizontal stress in respect of the stability and performance of roadway voids. Horizontal stress impacts can be particularly sensitive to interaction between roadways in the same seam.

Figures 3.2 and 5.4 illustrate conceptually how lateral stress shadows develop about the flanks of a roadway that has penetrated a horizontal stress field. No fixed distance can be assigned to the widths of these stress relief zones since they vary with geology, the in situ stress field magnitude and direction, roadway dimensions, and the extent of structural disturbance to the immediate roof and floor strata. Therefore, design needs to be supported with analytical and/or numerical analysis and field monitoring.

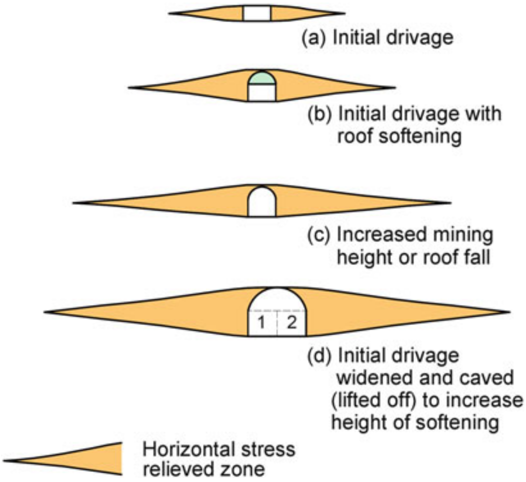

(a) Initial drivage
(b) Initial drivage with roof softening
(c) Increased mining height or roof fall
1
2
(d) Initial drivage widened and caved (lifted off) to increase height of softening
Horizontal stress relieved zone

**Fig. 5.4** Illustration of the manner in which the widths of horizontal stress relieved zones increase with increase in ‘height of softening’

The term **height of softening** is often used in relation to the height of disturbance of the immediate roof strata. There is no unique definition for this term. It is described generally as the height of delamination of the immediate roof strata, with this height being based on criteria such as a threshold value for parting width (for example, 3 mm) or a threshold value for vertical strain value (for example, 0.5 % in some cases and 0.75 % in others). Irrespective of the limit criteria, the term itself is apt when dealing with horizontal stress since it reflects engineering mechanics in that softening of the immediate roof strata results in horizontal stress being transferred higher in the roof strata. In turn, this increases the lateral extent of the horizontal stress relieved zones flanking the affected roadway as shown in Fig. 5.4.

If minimum pillar width requirements allow, a stress shadow associated with the redirection of in-seam stress and the formation of a zone of softening (discussed in Sect. 3.4) can be utilised to advantage to drive a subsequent flanking roadway in a reduced horizontal stress environment, illustrated conceptually in Fig. 5.5. The solid lines depict the pre-mining trajectory of the major horizontal stress in the immediate roof at

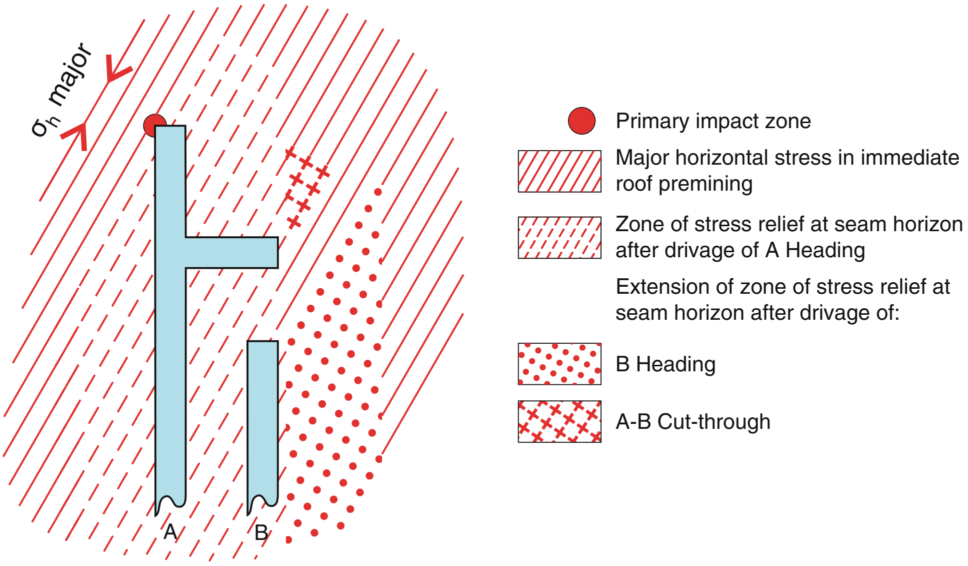

$\sigma_h$ major
A
B
● Primary impact zone
Major horizontal stress in immediate roof premining
Zone of stress relief at seam horizon after drivage of A Heading
Extension of zone of stress relief at seam horizon after drivage of:
B Heading
A-B Cut-through

**Fig. 5.5** Roadway being driven close to an adjacent roadway in order to take advantage of a horizontal stress relieved zone

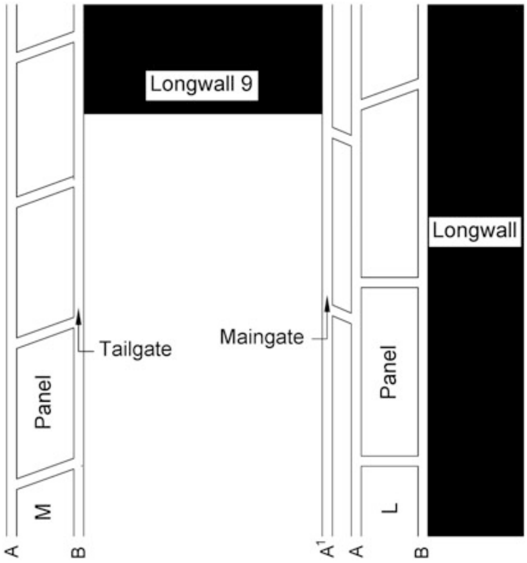


**Fig. 5.6** The concept of a ‘sacrificial gateroad’ as employed in a high horizontal stress environment at Appin Colliery, Australia, to create a stress shadow in which to subsequently drive the maingate (Modified from Todd 1983)

extraction height. The dashed lines depict a zone of stress relief at the seam horizon after the drivage of A Heading. This stress shadow is due to the combined effects of stress within the mining horizon being redirected into the roof and floor strata and roof softening forcing the horizontal stress higher up into the immediate roof. The dotted and crossed lines depict extensions of these zones of stress relief associated with the drivage of B Heading and A-B Cut-through, respectively. The major stress relief benefit comes from the impact of roof softening since coal seams are not generally subjected to high levels of horizontal stress (see Sects. 2.6.7 and 3.4).

In some instances, the immediate roof of the initial roadway is deliberately allowed to deteriorate in order to increase the height of softening before developing any flanking roadways. Figure 5.6 shows an example of this principle in practice. After the passage of the longwall, a new maingate ($A^1$) was driven in the stress relief shadow of the road (A) that would normally have constituted the maingate. Alternative attempts to achieve stress relief by cutting a slot in the roof of an advancing roadway close to the working face have met with mixed success. The approach results in regular interruptions to face operations such that, in a production environment, sacrificing a roadway and driving a new roadway may be a more efficient method of treatment.

The most effective means for controlling ground conditions around an excavation is to restrict the span of the excavation, as evidenced in Fig. 2.42 and by Eq. 2.48 which show that bending stress and Euler buckling load are a function of the square of the span. However, sometimes relatively wide spans are unavoidable, usually in order to accommodate large items of equipment. This is the situation in the case of a longwall face installation roadway, which typically ranges from 7 to 11 m in width. Because longwall gateroads have a longer and more critical service life than a longwall face installation roadway, they usually take precedence in being orientated in an optimum direction relative to the major horizontal stress direction. Other factors, such as seam dip and lease boundaries, may also result in longwall face installation roadways not being favourably orientated in respect of horizontal stress. In these situations a roadway may be developed and deliberately softened and sacrificed, as shown in Fig. 5.4d, in order to precondition the site of the installation face road. The concept and effectiveness of these so-called **sacrificial roadways** is noted in publications (for example, Galvin 1996; Doyle and Gale 2004) and discussed in more detail in Sect. 9.6.

In many high horizontal stress situations, factors such as the need to ensure pillar stability, to maintain serviceable roadways, and to limit surface subsidence do not make it safe, practical or productive to take advantage of stress relief shadows by driving roadways close together. This not only affects drivage conditions but can give rise to primary impact zones when holing into a cut-through (Fig. 5.7a), or if a roadway has to be back-holed (Fig. 5.8). In the situation shown in Fig. 5.7a, the impacted side of the face of the cut-through and that of B Heading coincide, while opposite sides of the respective

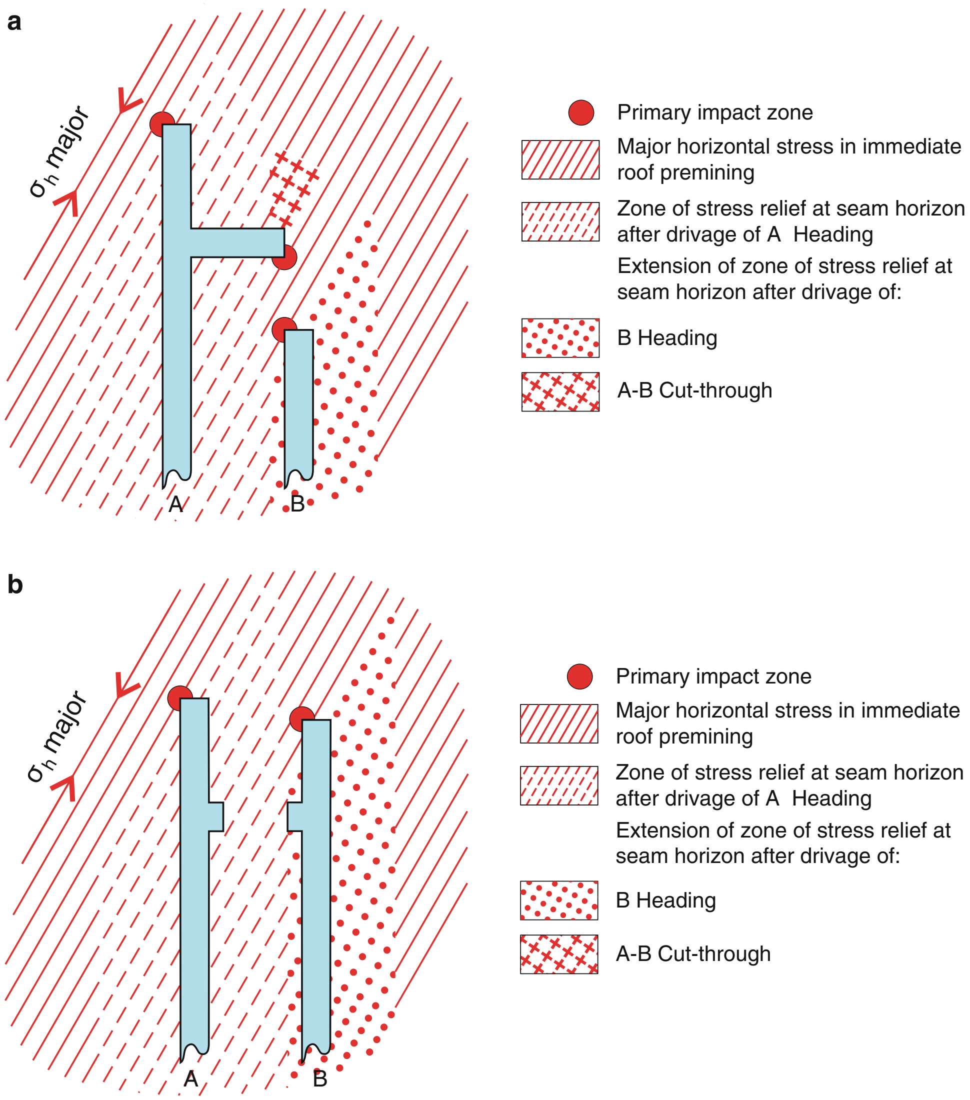

a
σh major
A
B
Primary impact zone
Major horizontal stress in immediate roof premining
Zone of stress relief at seam horizon after drivage of A Heading
Extension of zone of stress relief at seam horizon after drivage of:
B Heading
A-B Cut-through
b
σh major
A
B
Primary impact zone
Major horizontal stress in immediate roof premining
Zone of stress relief at seam horizon after drivage of A Heading
Extension of zone of stress relief at seam horizon after drivage of:
B Heading
A-B Cut-through

**Fig. 5.7** Stress notching associated with forming cut-throughs orientated at an angle to the major horizontal stress direction. (**a**) Driving towards a partially driven cut-through. (**b**) Overdriving prior to forming a cut-through

faces are subjected to stress points (or notches) for the situation shown in Fig. 5.8.

The situation shown in Fig. 5.7a is particularly adverse because the holing point where the two stress points interact coincides with the site of an intersection. Figure 5.7b illustrates how stress points can be avoided when driving a cut-through by overdriving the headings a distance sufficient to precondition the cut-through site by placing it in a stress shadow.

The need to back-hole a heading usually arises due to the advance of a lagging heading being constrained by a geological structure, poor ground conditions, or equipment breakdown. Figure 5.8 illustrates the situation for a two heading panel development orientated at an angle of

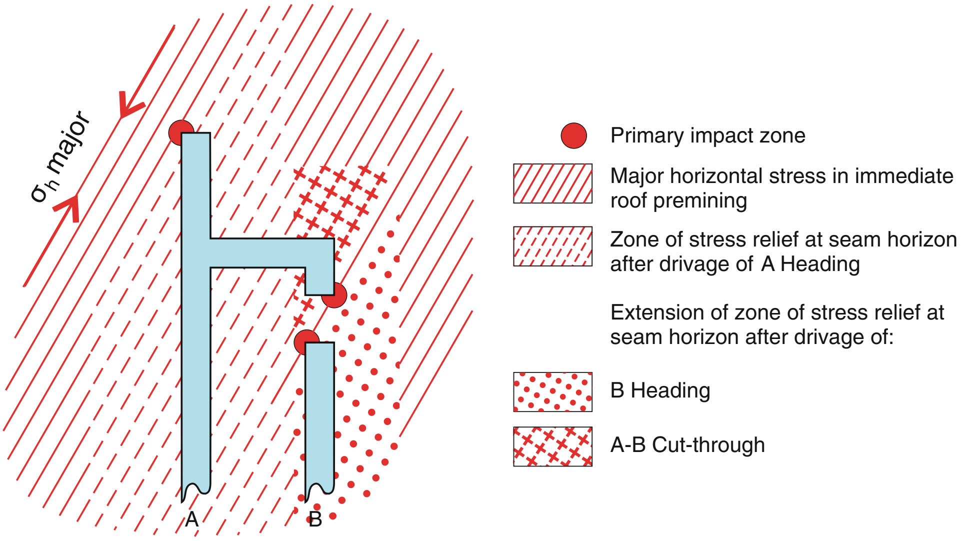

σh major
Primary impact zone
Major horizontal stress in immediate roof premining
Zone of stress relief at seam horizon after drivage of A Heading
Extension of zone of stress relief at seam horizon after drivage of:
B Heading
A-B Cut-through
A
B

**Fig. 5.8** Stress notching associated with back-holing a roadway orientated at an angle to the major horizontal stress direction

about 30° to the major horizontal stress direction. The final stages of the back-holing can be associated with very poor roof, face and floor conditions because stress points occur on opposite sides of the roadway at the hole-through site. The intervening pillar has been known to disintegrate with some 3 m still to go to hole through (Galvin 1996). This has the effect of effectively increasing the unsupported cut-out distance, so aggravating the development of a roof fall at the holing site.

In these circumstances, it is important not to back-hole at the location of a proposed intersection and to maintain ground support very close to the face and, preferably, over and ahead of the face. Angled cable bolts and face spiling (see Chaps. 6 and 7) can find application in these circumstances. If using a two pass continuous miner, it can be advantageous to limit roadway width by only mining one pass over the maximum safe and practical distance until the hole-through is achieved. Additional benefit may be obtained by taking this cut from the centre of the roadway. Since the strength of highly stressed rock can decrease over time, speed is of the essence. While it is likely that little can be done to speed up cutting rates in the circumstances, it is important that the face is not allowed to stand in the final stages of the hole-through.

### 5.2.4 Panels

The principles depicted on a local scale in Figs. 5.4, 5.5, 5.6, 5.7, and 5.8 regarding horizontal stress distribution about a roadway also apply on a regional scale to horizontal stress distribution about a series of mining panels. The main difference between their application on a local scale and on a regional scale relates to the height of softening, which is much greater around a panel and, therefore, allows the concept of a ‘bow wave’ to be applied to stress redistribution. The depth of mining and the nature of subsidence over a panel determine the extent to which horizontal stress can continue to be transmitted through the roof strata.

Figure 5.9 illustrates how goaves function as a stress relief slot on a regional scale. In this example, each extraction panel commences in the stress shadow of the preceding panels. As mining approaches the end of the extraction panel, it progressively moves out of the stress shadow and into the regional horizontal stress field,

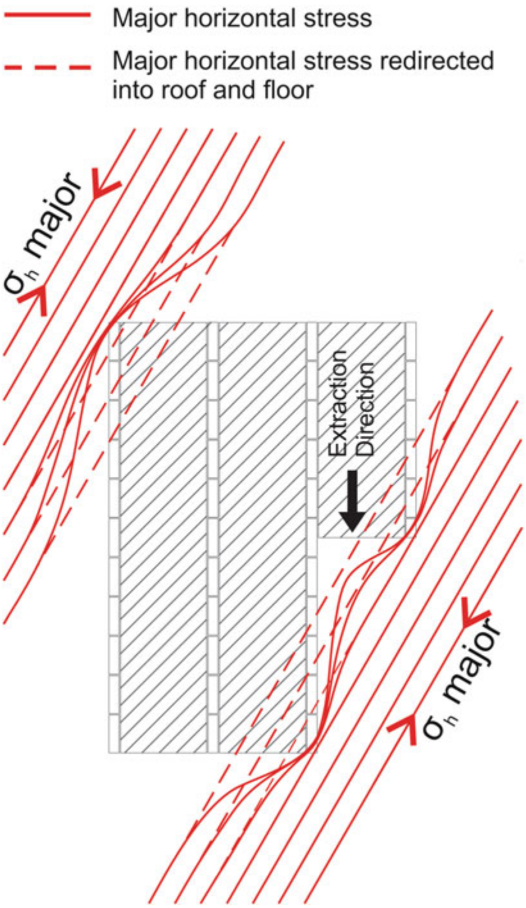

Major horizontal stress
Major horizontal stress redirected into roof and floor
σ_h major
Extraction Direction
σ_h major

**Fig. 5.9** Regional stress relief about a series of total extraction panels when mining from within to outside of the stress shadow created by the goaves

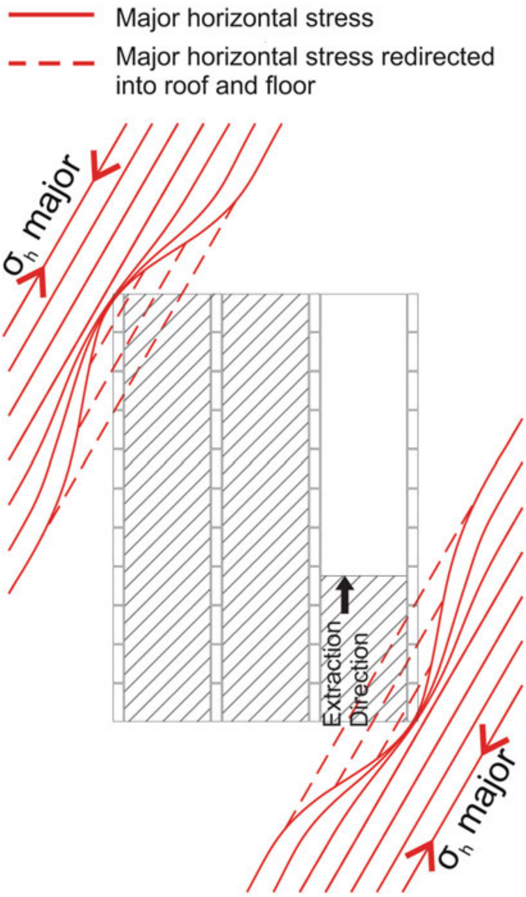

Major horizontal stress
Major horizontal stress redirected into roof and floor
σ_h major
Extraction Direction
σ_h major

**Fig. 5.10** Regional stress relief about a series of total extraction panels when mining from outside to within the stress shadow created by the goaves

resulting in a stress notch developing at the panel development face (or maingate face if this were a longwall layout). The notch is present for the remaining life of the panel. In the case of longwall mining, the disadvantage of this stress notch has to be balanced against the benefit of being able to drive the installation face in a stress shadow.

The situation is quite different if the panels shown in Fig. 5.9 were to be extracted from the opposite direction (Fig. 5.10). A longwall installation face would not only be subjected to the major horizontal stress but also to a stress notch adjacent to the corner of the previously extracted panel. Goaf formation takes place in an environment that is subjected to increasing horizontal stress, as opposed to a stress relieved environment in the former case. Initially, a stress point may begin to form at the maingate end but this dissipates once caving develops. Thereafter, the maingate end face is protected from horizontal stress by the adjacent goaf.

The regional horizontal stress behaviour depicted in Figs. 5.9 and 5.10 can give rise to a double stress notch effect when an extraction panel retreats past the end of an adjacent extraction panel, as shown in Fig. 5.11. The situation is analogous to back-holing a roadway but far more adverse because the area is also subjected to two vertical abutment stress fronts. Further complicating factors in the case of longwall mining are, firstly, that support of the gateroads

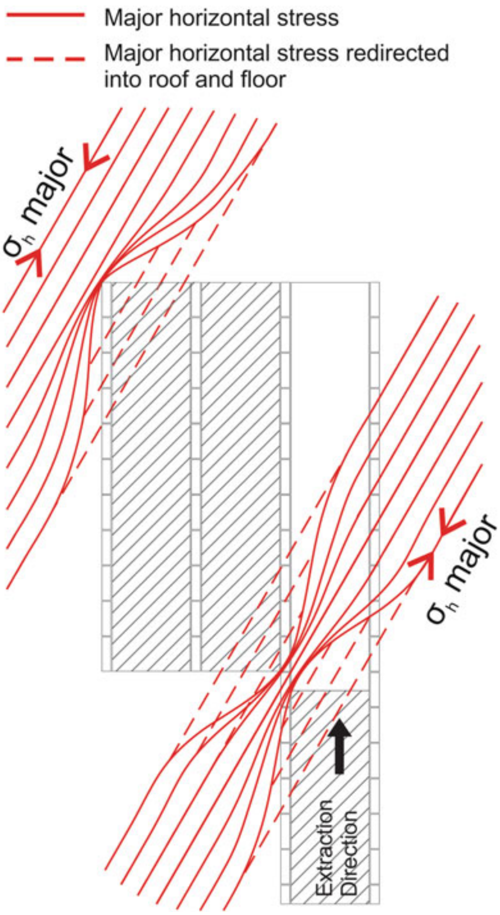


**Fig. 5.11** Double stress notch effect created when a total extraction panel retreats past the end of an existing total extraction panel

in this region has to be undertaken in a manner that does not impede the subsequent passage of the longwall face equipment and, secondly, floor heave rather than roof displacement may prove the greater impediment to advancing the longwall face through the area.

Secondary roadway support in the area should be installed well in advance of the longwall face, with its selection and placement having regard to how far the shearer needs to cut into the tailgate in order to advance the longwall face. Longwall face creep needs to be very carefully controlled to prevent roadway support being damaged or dislodged by the ingress of the powered supports into the gateroad. Rate of extraction should be maximised when working through the zone of interaction and the face should not be slowed down until some distance past the notch point. This is because the fractured ground is prone to unravel when confinement is reduce as a result of ground pressure being relieved.

### 5.2.5 Interaction Between Roadways and Excavations

Given that a roadway formed other than parallel to the major horizontal stress direction affords some stress relief to an adjacent excavation, it might be thought that a caved and subsided region (goaf) would provide a greater and more laterally extensive zone of horizontal stress relief to an adjacent roadway, for example, a longwall tailgate. However, this is not always the case, for reasons that become apparent when the relative size and locations of the excavations are drawn to scale.

A scaled example is illustrated in Fig. 5.12 for the case of a 6 m wide, 3 m high tailgate roadway separated from a 300 m wide longwall panel by 30 m wide chain pillars at a depth of 300 m. Primitive horizontal stress is assumed to be twice the vertical primitive stress. The figure shows a conceptualisation of how horizontal stress is redistributed around the zone of softening, or soft inclusion, that develops over the goaf as a result of caving, fracturing and bed separation. The zone of softening is assumed to extend to a height of half the longwall panel width and to have no capacity to transmit horizontal stress. It becomes obvious from the conceptualisation that, in fact, circumstances could arise where the chain pillar and the surrounding strata including the roof and floor strata of the tailgate are subjected to resultant stresses that have a high component of lateral stress. Consequently, the chain pillar and tailgate will also be impacted by shear stresses.

Each situation should be assessed in its own right, with consideration being given to utilising numerical modelling to predict principal stress magnitudes and distributions. The situation depicted in Fig. 5.12 has been adapted to a two-dimensional elastoplastic finite element model to demonstrate the insight that numerical

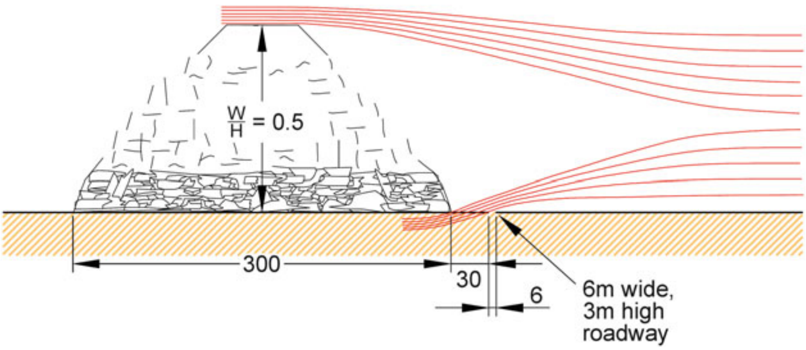


**Fig. 5.12** Conceptualisation of the deviation of horizontal stress around a softened goaf inclusion, showing the location and size of a tailgate drawn to scale and its susceptibility to being impacted by lateral and shear stresses

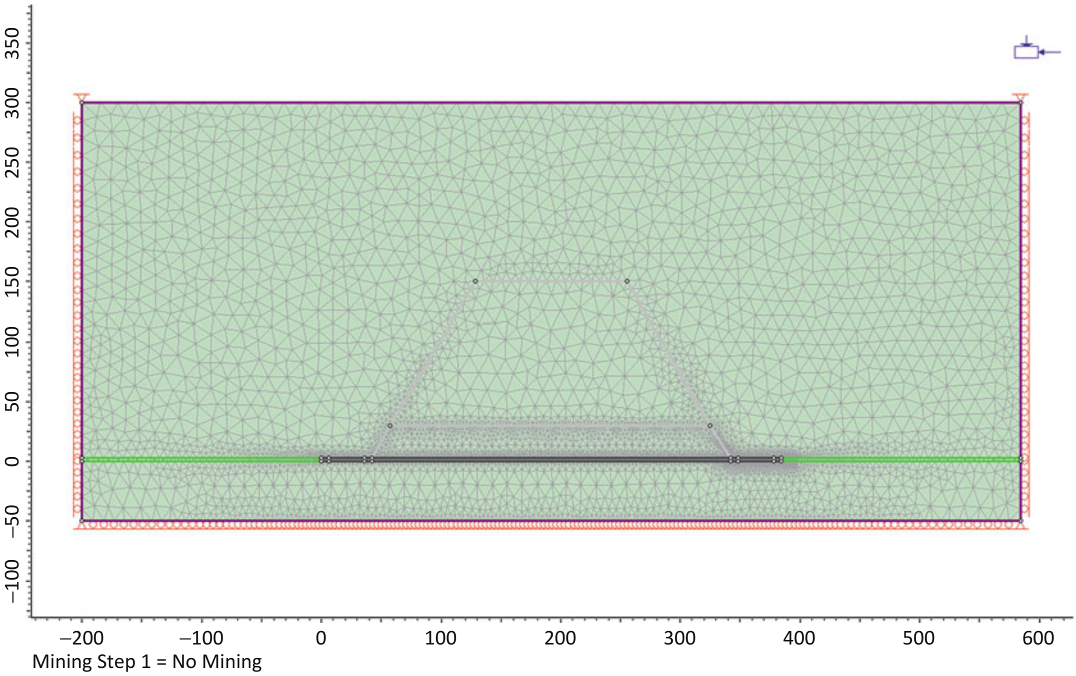
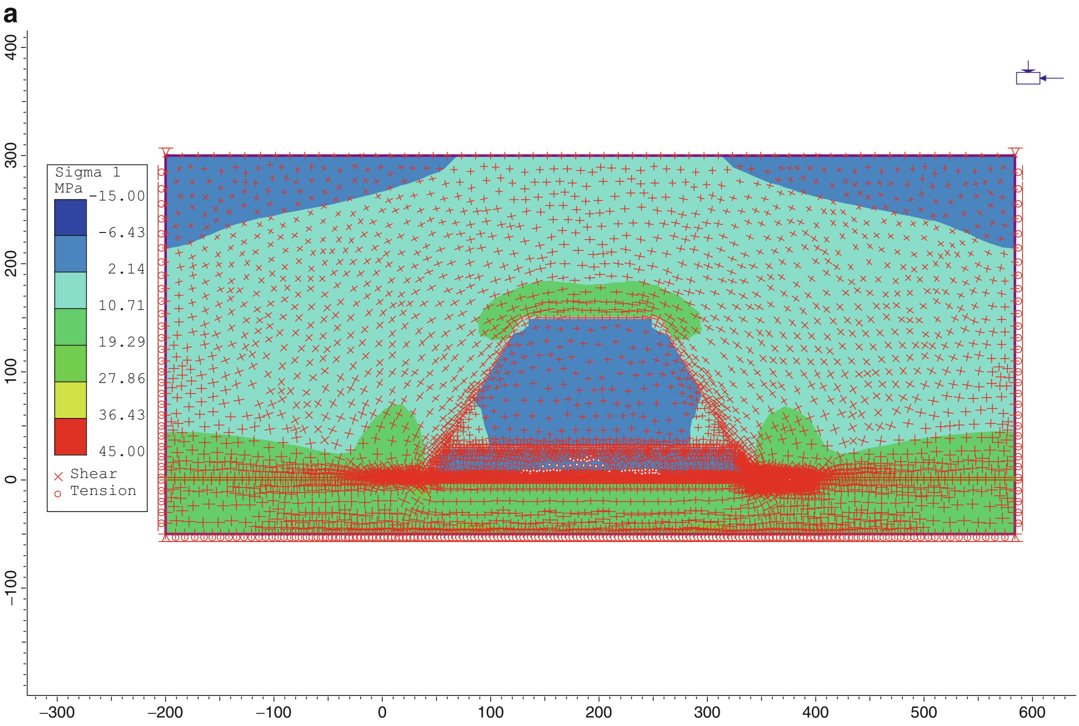


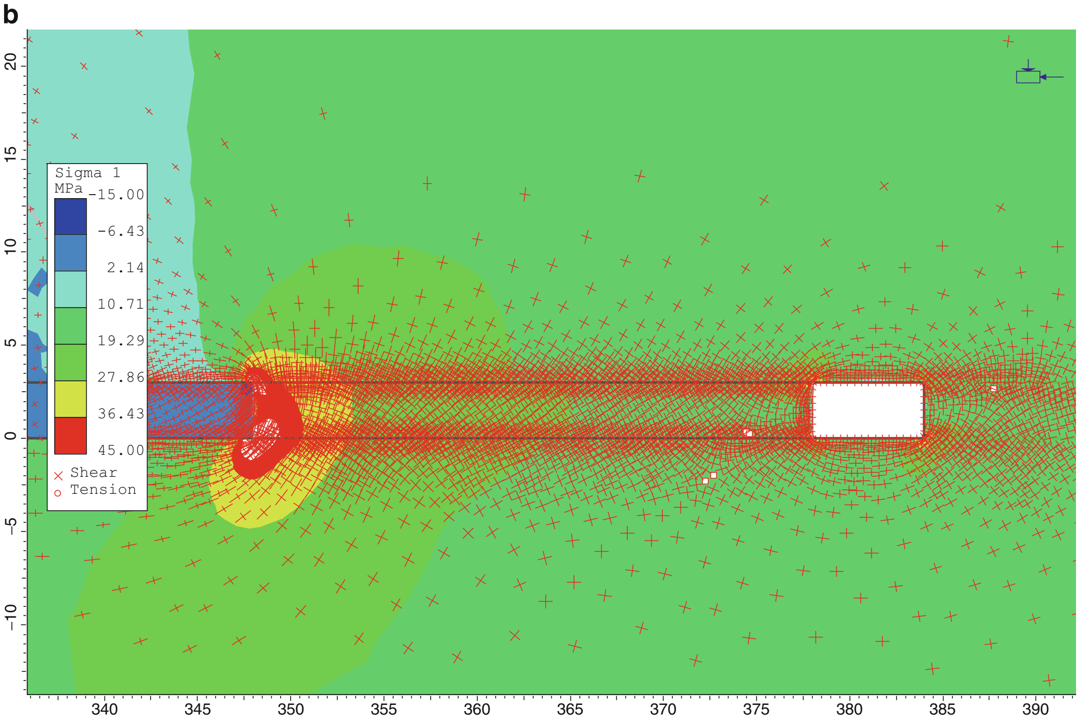
**Fig. 5.13** Construct of the numerical model used to investigate principal stress magnitudes and distributions about a chain pillar subjected to single abutment loading

modelling can provide in these types of situations.

The construct of the numerical model is shown in Fig. 5.13. The sides and base of the model comprised roller contacts. The horizontal to vertical stress ratio was set at 2 and the Mohr-Coulomb failure criterion was invoked, with all materials being assigned a peak friction angle of 35° and a peak cohesion of 10.5 MPa. The strata above the goaf was simulated by three zones of differing modulus to reflect caved material, fractured material and constrained material. The caved and fractured zones were modelled as body-force materials, so that they are only loaded by their own weight. This results in the soft inclusion having a capacity, albeit limited, to transmit lateral stress. The overall magnitude and distribution of principal stress predicted by the model are presented in Fig. 5.14a and shown in a magnified form in



Step 4 Maingate Principal Stress Trajectories



Step 4 Maingate Principal Stress Trajectories

**Fig. 5.14** An example of how numerical modelling can be utilised to gain insight into stress distribution around pillars and roadways adjacent to caved and subsided strata. (**a**) Predicted principal stress distribution and magnitude throughout the modelled region. (**b**) Predicted principal stress distribution and magnitude in and around the chain pillar and tailgate roadway roof and floor.

and around the chain pillar and tailgate area in Fig. 5.14b.

The numerical modelling shows how the horizontal stress trajectories flow around the soft inclusion. It predicts that the magnitude of principal stress will reduce in the chain pillar and around the tailgate roadway as soon as caving and subsidence are initiated but, nevertheless, the roof of the tailgate will remain in compression. Maximum stress relaxation occurs some distance above the seam horizon. At the bottom corner of the longwall panel there is a concentration of horizontal stress trajectories indicative of high horizontal stress.

The modelling illustrates some of the complexities associated with pillar system design. For example, it shows that the formation of a goaf can result in a significantly different loading environment to the predominantly vertical principal stress environment associated with the derivation of empirical pillar strength formations such as those of Salamon and Munro (1967), Bieniawski (1983), Mark and Bieniawski (1987), and Salamon et al. (1996). The potential for this to result in a different mode of pillar behaviour should be borne in mind if the chain pillar system design is reliant on empirically derived pillar strength formulations. In the modelled case, the elevated lateral component of stress may actually enhance pillar confinement and, therefore, strength.

## 5.3 Multiseam Workings

### 5.3.1 Framework

Primary ground engineering factors that should be considered when assessing interaction between multiseam workings include:

- existing stress magnitudes and distributions in mined seams, including areas of stress relief;
- the rate that stress in the mining horizon of one seam dissipates with distance into the roof and floor of that seam;
- the nature and extent of structural damage, fracture networks and subsidence experienced by a targeted unmined seam and its roof and/or floor interburdens (partings);
- the capacity of undermined strata to continue to transmit shear stress;
- the location and severity of zones of flexure associated with the flanks of sub-surface subsidence basins;
- the orientation of a proposed working face relative to the orientation of mining-induced fracturing associated with the prior extraction of an adjacent seam; and
- the location and severity of zones of flexure associated with the flanks of sub-surface subsidence basins.

Strata failure in multiseam situations can take a number of forms, the more common being extensive roof falls, a collapse of coal pillars, or a collapse of one or more excavations. Careful consideration has to be given to consequential risks associated with strata failure. These include windblast and inrush of noxious and/or flammable gases, fluids, and materials that flow when wet or unconfined.

There are no unique criteria for defining what constitutes successful or unsuccessful multiseam mining. The degree of interaction that can be tolerated is determined by safety, environmental and economic considerations.

### 5.3.2 Pillar Systems

Figure 5.15 illustrates conceptually how vertical stress at seam level for the non-caving situation depicted in Fig. 5.1 (curve ‘a’) dissipates with distance into the roof or floor strata. Curve ‘b’ shows the situation a relatively short distance into the roof. Partial stress relief has occurred over the central portion of excavation, but as the alignment of the excavation abutment is approached, the stress passes back through its primitive level and remains elevated for some distance into the abutment. Similar to the profile depicted by curve ‘a’, the area of zone $A_2$ equals the area of zone $B_2$. Some relatively short distance further into the roof, the induced vertical

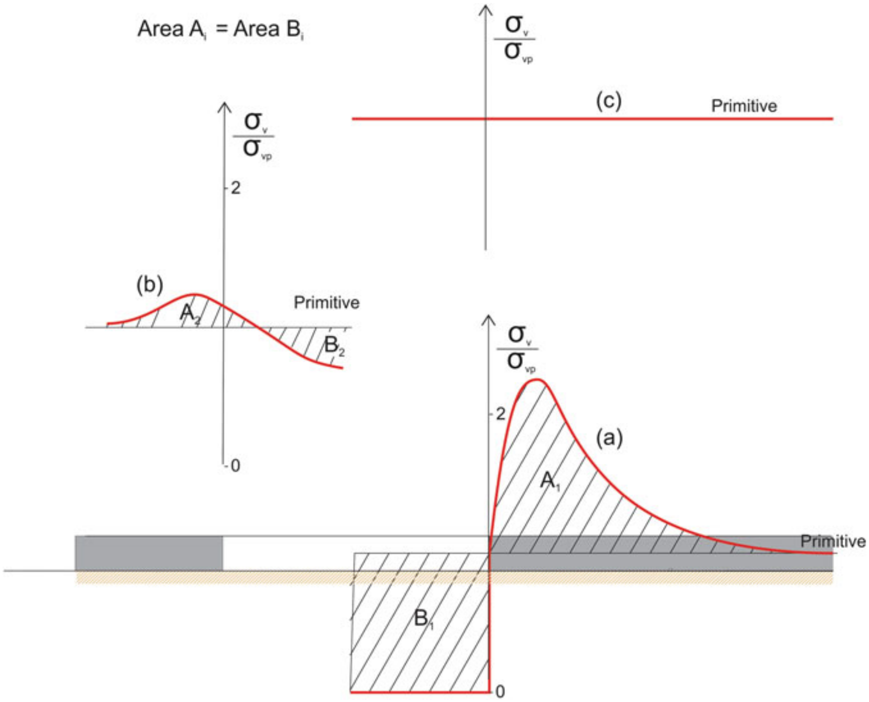

Area $A_i$ = Area $B_i$
$\frac{\sigma_v}{\sigma_{vp}}$
(c)
Primitive
$\frac{\sigma_v}{\sigma_{vp}}$
2
(b)
$A_2$
Primitive
$B_2$
0
$\frac{\sigma_v}{\sigma_{vp}}$
2
(a)
$A_1$
Primitive
$B_1$
0

**Fig. 5.15** Conceptualisation of how vertical stress about an isolated excavation is dissipated with increasing distance into the roof and floor for a non-caving situation

stress along a horizontal plane will have dissipated and, for all practical purposes, be equal to primitive stress, as depicted by curve ‘c’.

The dissipation of vertical stress above or below mine workings and the impact that this stress has on adjacent seams is a function of the thickness and competence (strength and structural integrity) of the interburden. Figure 5.16a shows an often quoted example of the distribution and dissipation of vertical stress about bord and pillar workings as determined by elastic finite element analysis for a very shallow case where pillar width is equal to 0.8 times bord width. For this regular bord and pillar layout, the field stresses have almost returned to their primitive state at a distance of 0.75 times the pillar centre distance above and below the seam.

The situation is more complicated in the vicinity of a solid abutment or an interpanel or barrier pillar due to the change in the stiffness of the loading system and the effect of gravity (Fig. 5.16b). In particular, the roof is exposed to much higher displacements than the floor and the influence of an interpanel or barrier pillar diminishes less rapidly with vertical distance from the seam. It is relatively simple and quick to produce more sophisticated and quite accurate numerical simulations of these situations.

Figure 5.17 shows some of the possible scenarios associated with multiseam bord and pillar workings. These include:

- smaller pillar over larger pillar, case A;
- pillar over bord, case B; and
- pillar partially over a bord and partially over a pillar, case C.

The acceptability of these types of scenarios is a function of the magnitude of the vertical stresses and the thickness and competence of the interburden. There are three tiers of intervention for managing interaction between multiseam workings that rely on pillars for stability. If the interburden is relatively thin and/or weak, both panel and interpanel pillars will need to be superimposed (stacked or columnised) to avoid

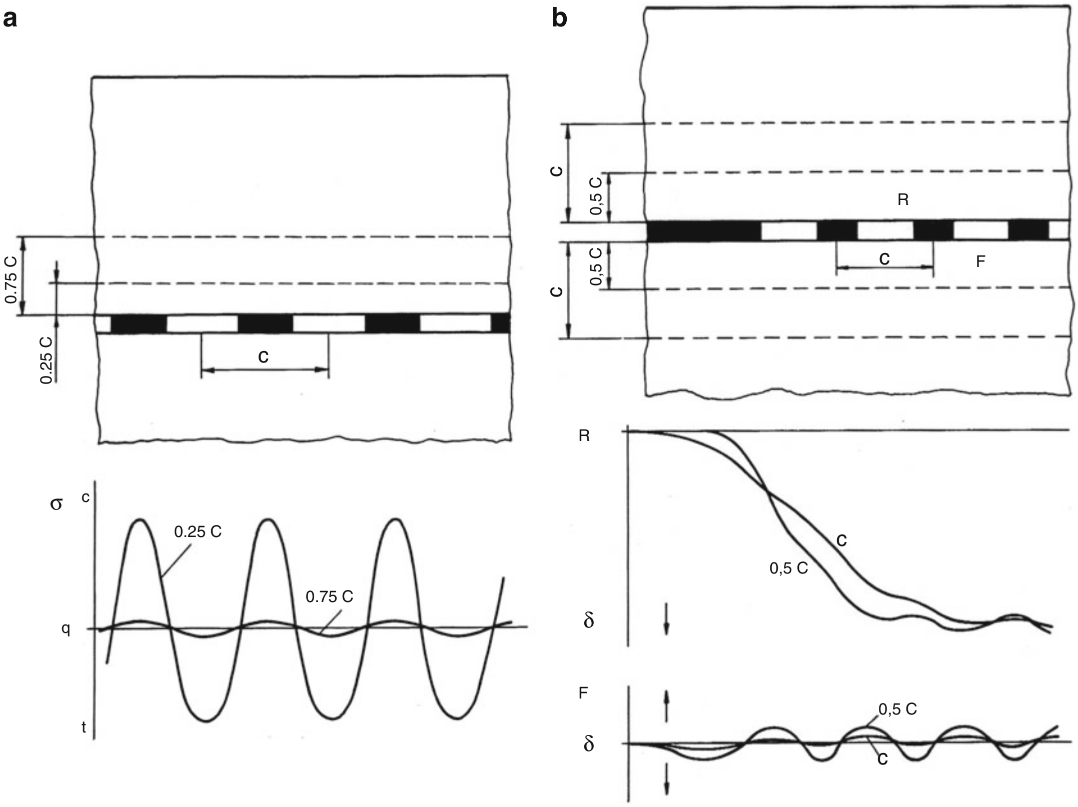


**Fig. 5.16** Distribution of vertical stress and displacement about bord and pillar workings (After Salamon and Oravecz 1976). (**a**) Vertical stress distribution. (**b**) Displacement distribution

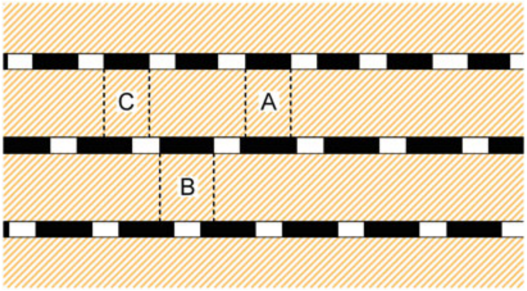


**Fig. 5.17** Examples of scenarios associated with multiseam bord and pillar workings

the pillars punching into the surrounding excavations. As interburden thickness and/or strength increase, the need to superimpose panel pillars falls away but remains for interpanel pillars. With further improvement in interburden thickness and competence, the need to superimpose both pillar types can be dispensed with.

The quality of the interburden becomes more important as it becomes thinner. Salamon and Oravecz (1976) noted at the time that it was thought the mining of neighbouring bord and pillar workings is only possible when the interburden thickness is less that 0.3–0.5 times the bord width if it contains a reasonable proportion of competent sandstone. They advised that if the interburden thickness is less than 1.5–2 times the bord width, the possibility of failure of the interburden should be taken into account. It was suggested that in addition to superimposing the workings in these circumstances, the Salamon and Munro safety factor of the workings in each seam should not be less than 1.8 and that the safety factor of hypothetical pillars having a height equal to the combined height of the workings should not be less than 1.4.

In the absence of numerical modelling to better quantify the potential extent and severity of

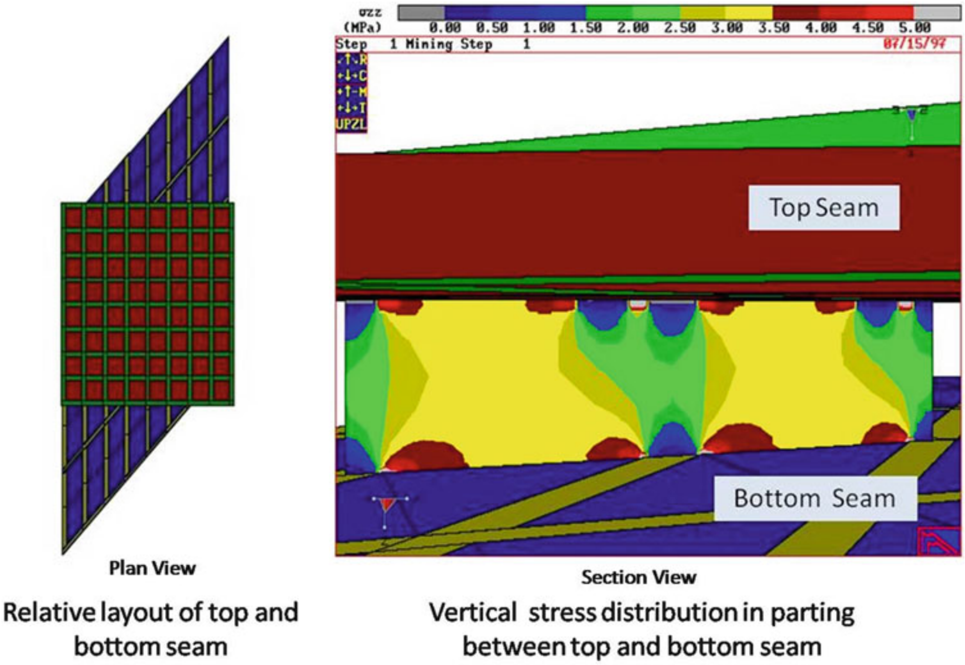


**Fig. 5.18** Evaluation using a three-dimensional elastic numerical model of vertical stress distributions and magnitudes above and below pillars in a multiseam mining situation in which the pillars are not superimposed (After Galvin 1997a)

interaction between multiseam workings, superpositioning of workings is the most effective risk mitigation measure in near-horizontal seams. The situation is more complex in the case of dipping seams, where allowance has to be made for non-symmetrical pillar stress profiles and offset stress profiles between seams (see Sect. 11.5). When pillars need to be superimposed, the dimensions of the mine layout in each seam are dictated by those of the seam with the largest pillars in plan. Depending on factors such as seam height, coal quality, geological conditions, and optimisation of percentage extraction, these pillars may not necessarily occur in the deepest seam. As a very small difference in survey bearing between seams or an error in traverse runs can quickly lead to bords and pillars not being fully superimposed, it is advisable to periodically verify that the workings are properly superimposed by, for example, surveyed interseam boreholes.

Caution is advised if applying design recommendations of the past as some are flawed. In light of advances in numerical modelling, it is advisable to utilise this tool to evaluate displacement and load distributions about multiseam bord and pillar workings as a basis for deciding whether or not to superimpose bord and pillar workings. Figure 5.18 shows an example of the output from a three-dimensional elastic model utilised for this purpose. An elastic model was adequate in this case because of the low stress regime. However, in many cases it may be more appropriate to utilise an inelastic model in order to properly evaluate both stress path and the nature and extent of the associated strata deformation.

### 5.3.3 Extraction Panels

Three aspects, in particular, warrant careful consideration when assessing the potential for interaction between total extraction workings in adjacent seams, and the impacts that may arise from this interaction. These are the capacity for interburden and overburden to transmit shear stress; vertical stress profiles at each seam horizon; and shear stress concentrations at an active mining face. A conceptual four zone model of how sub-surface subsidence develops over a total extraction panel is illustrated in Fig. 3.5. The extent to which these zones develop determines the degree to which the subsided strata can continue to transmit shear stress. This, in turn, influences the magnitude and distribution of vertical stress at the mining horizon and the magnitude of surface subsidence.

Figure 5.19a shows a conceptual depiction of vertical stress distribution and magnitude at mid-seam height in each of two seams where the upper seam has already been totally extracted and the lateral distance between the face of the lower seam and the upper seam abutment is still quite considerable. The vertical stress profile distribution in the upper seam mirrors that shown in Fig. 3.6 for a total extraction panel in a single seam. Figure 5.19b depicts the same situation but now shows how abutment stress is distributed in the roof and floor strata of the upper seam.

The face abutment in the lower seam is effectively in destressed ground due to the caving of the superincumbent strata during extraction of the upper seam. The overhanging wedge of interburden at the face of the lower seam accounts for a marginal decrease in vertical stress acting on the floor at this point and a marginal increase in face abutment stress. There is a larger decrease in vertical stress acting on the floor of the lower seam towards the abutment of the upper seam because it is protected from vertical load by the over-hanging strata in the upper seam. This decrease is balanced by an increase in vertical stress in the lower seam as it comes within the sphere of influence of the high abutment stress in the upper seam.

As the lower seam approaches the abutment of the upper seam, there is a further reduction in floor stress close to the face of the lower seam and an increase in abutment stress ahead of this face (Fig. 5.19c). Once the lower seam face is under the abutment of the upper seam, the stress profile in the lower seam mirrors that for single seam extraction under virgin ground shown in Figs. 3.6, 5.19a, 5.19b.

Hence, total extraction workings in a lower seam that are located beneath the goaf of total extraction workings in an upper seam are subjected to destressed conditions, the extent being dependent on the thickness and competence of the interburden. However, as these workings approach the abutment of the upper seam workings, abutment stress in the lower seam rises and ultimately plateaus at a magnitude and distribution equivalent to mining in virgin conditions.

The situation is quite different when the lower seam workings approach the upper seam workings from the solid rather than the goaf. In this case, face abutment stress in the lower seam increases as the overburden load is funnelled through a progressively narrower corridor (Fig. 5.20). Ultimately, the lower seam face has to negotiate what is effectively a very highly stressed remnant pillar, before then operating in destressed conditions beneath the goaf of the upper seam.

The development of fracturing around the abutments of a total extraction panel as mining proceeds is discussed in Sect. 3.2 and illustrated, for example, by microseismic monitoring outcomes shown in Fig. 3.16 and field observations shown in Figs. 3.4 and 3.11. Depending on interburden thickness and strength, in multiseam mining these fractures can constitute man-made faults with all the associated consequences. A critical issue in this respect is the relative orientation of the panels in each seam. Parallel face orientations can lead to situations where the pre-existing mining-induced fractures run parallel to the active working face, with major implications for roof control on the active mining face. However, changing direction to avoid this situation then results in mining having to proceed under remnant interpanel (chain) pillars.

Remnant pillars in panels previously extracted in neighbouring seams have a high potential to cause adverse interaction in multiseam mining situations because they act as stress raisers by impeding caving and behaving as punches. Their

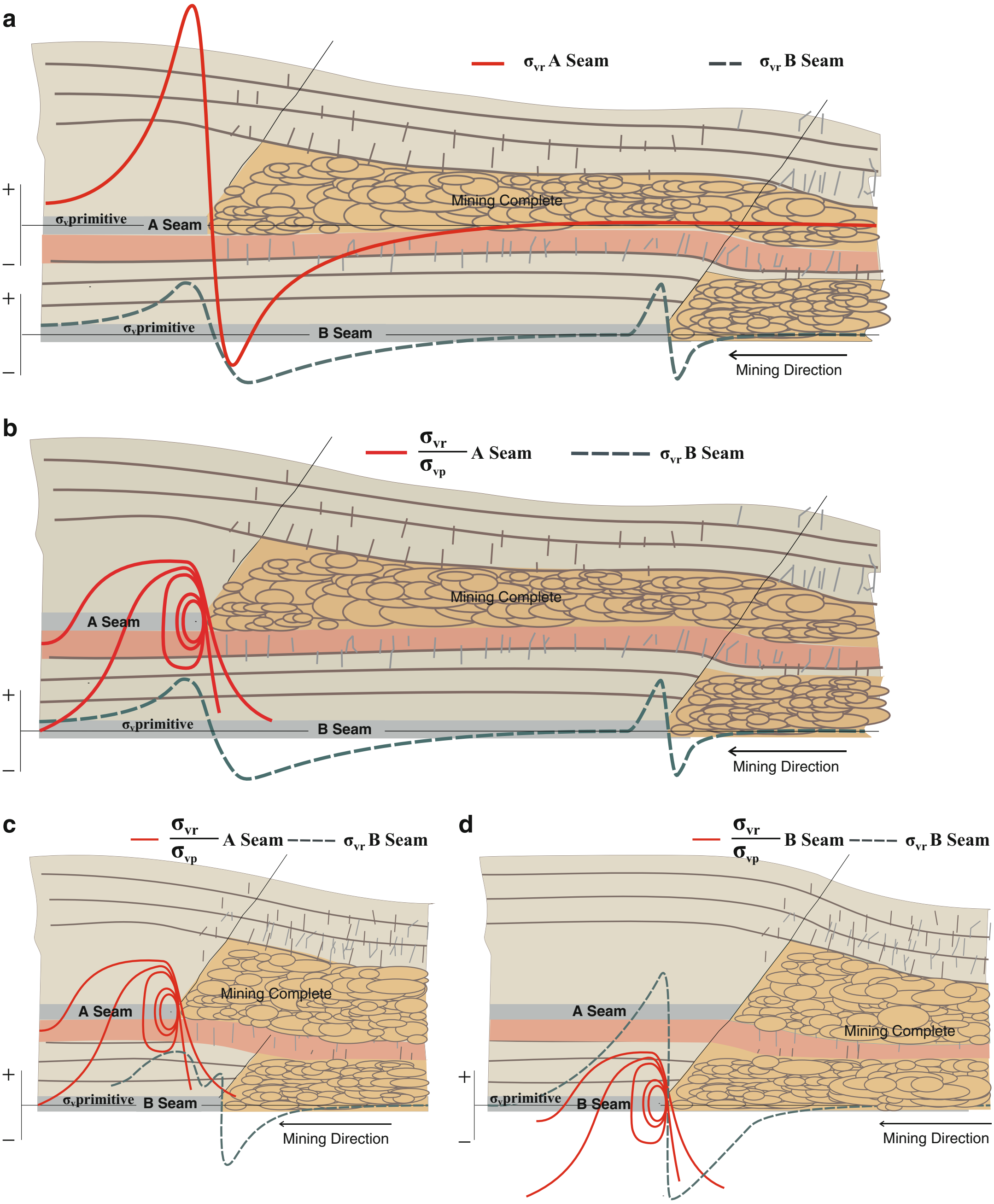

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**Fig. 5.19** Interaction between multiseam total extraction panels when lower seam approaches from under goaf. (**a**) Conceptual resultant vertical stress distribution and magnitude at mid seam height in upper seam and in lower seam. (**b**) Conceptual normalised vertical stress contours around upper seam and resultant vertical stress distribution and magnitude at mid seam height in lower seam. (**c**) Change in resultant vertical stress magnitude and distribution in lower seam as mining in this seam approaches the abutment of upper seam. (**d**) Once the lower seam face is under the abutment of upper seam, the resultant vertical stress profile in lower seam mirrors that for single seam extraction under virgin ground (as shown in (**a**) and (**b**) above)

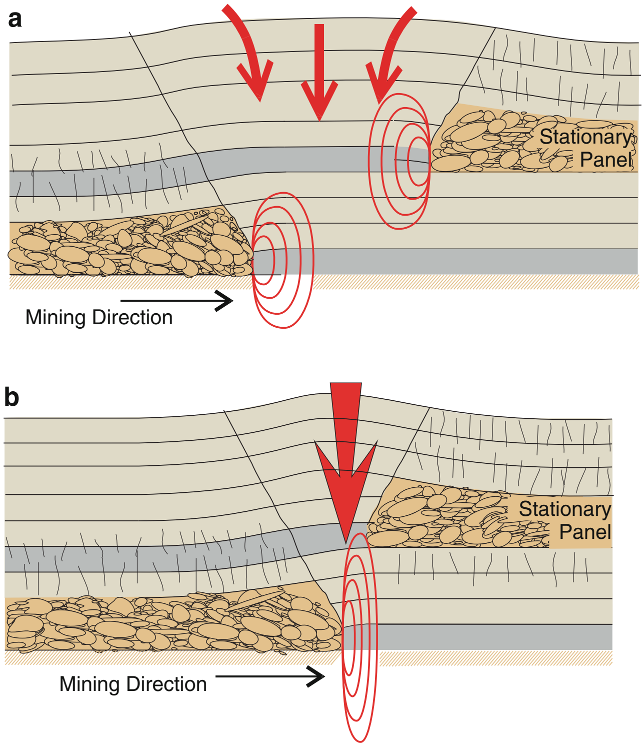


**Fig. 5.20** Interaction between multiseam total extraction panels when lower seam approaches from under solid. (**a**) When total extraction multiseam workings approach each other from opposite directions, abutment stress rises as the overburden load is channelled through a progressively narrower corridor. (**b**) Ultimately, the lower seam workings have to extract what is effectively a very highly stressed remnant pillar, before then operating in destressed conditions beneath the goaf of the upper seam.

impact is a function of the depth of mining, the mining layout in each seam, and the thickness and strength of the interburden. In general, if workings are not superimposed it is not unknown for remnant pillars to adversely impact mining operations in neighbouring seams when interburden thickness is less than 30 m. MacDonald (1997) concluded that a range of incidents associated with longwall mining 140 m beneath remnant pillars in Nova Scotia were strong indications of interaction. Microseismic investigations at Appin Colliery in Australia detected floor failure down to 120 m below longwall chain pillars at a depth of around 500 m (Kelly 2000). Numerical modelling studies by Lightfoot and Liu 2010 indicated that vertical stress concentrations under remnant pillars can persist for more than 200 m.

Chain pillars in longwall mining are a form of remnant pillar. There are three basic options for locating gateroads in multiseam longwall mining operations in order to manage the impact of these pillars, namely:

- Superimpose. The gateroads in each seam are aligned vertically.
- Indent. The gateroads in a lower seam are located under the goaves of the upper seam workings on the flanks of the upper seam interpanel pillars.
- Offset. The gateroads in the lower seam are located under the goaves of the upper seam workings, well away from the upper seam interpanel pillars.

Furthermore, there are two variations for superimposing gateroads and two for indenting gateroads. The first is associated with the gateroad of a longwall panel becoming the tailgate for the adjacent longwall panel, in which case the cut-throughs between gateroads have to be driven under the interpanel pillar of the upper seam, as illustrated in Fig. 5.21a. The second variant involves driving two new sets of gateroads for each longwall panel, one being a maingate set and the other a tailgate set, with the cut-throughs for each set of gateroads being driven under the goaf of the extracted panel in

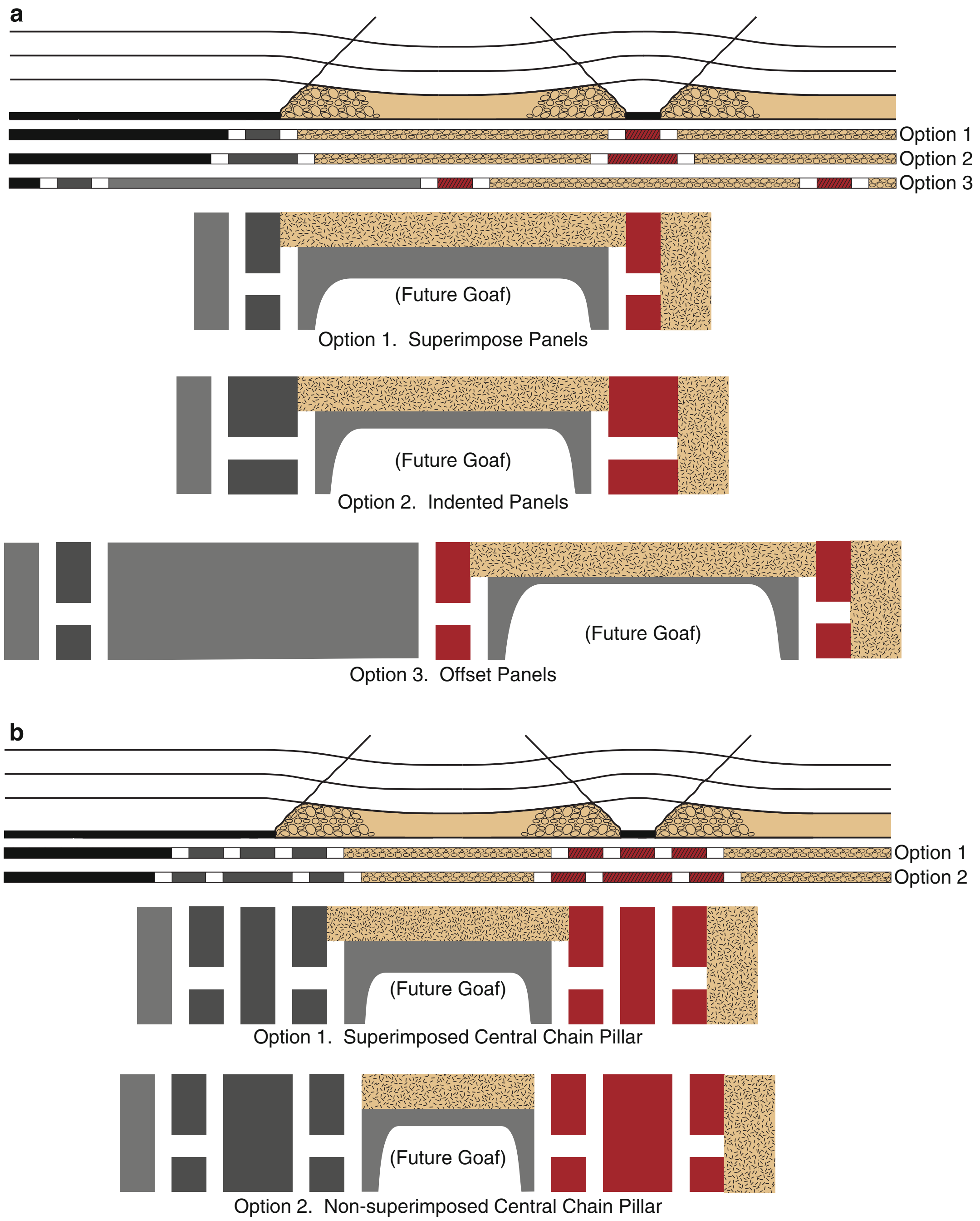

a
Option 1
Option 2
Option 3
(Future Goaf)
Option 1. Superimpose Panels
(Future Goaf)
Option 2. Indented Panels
(Future Goaf)
Option 3. Offset Panels
b
Option 1
Option 2
(Future Goaf)
Option 1. Superimposed Central Chain Pillar
(Future Goaf)
Option 2. Non-superimposed Central Chain Pillar

**Fig. 5.21** Options for locating gateroads in multiseam longwall mining. (a) Superimposed and indented gateroads connected by cut-throughs subjected to abutment stress. (b) Superimposed and indented gateroads connected by cut-throughs driven in destressed conditions beneath the goaves of workings in the overlying seam

the upper seam, illustrated in Fig. 5.21b. The width of the pillar separating each set of gateroads for the option shown in Fig. 5.21b is determined by the magnitude and distribution of the stress concentration beneath the inter-panel pillars of the upper seam and, hence, by the thickness and competence of the interburden.

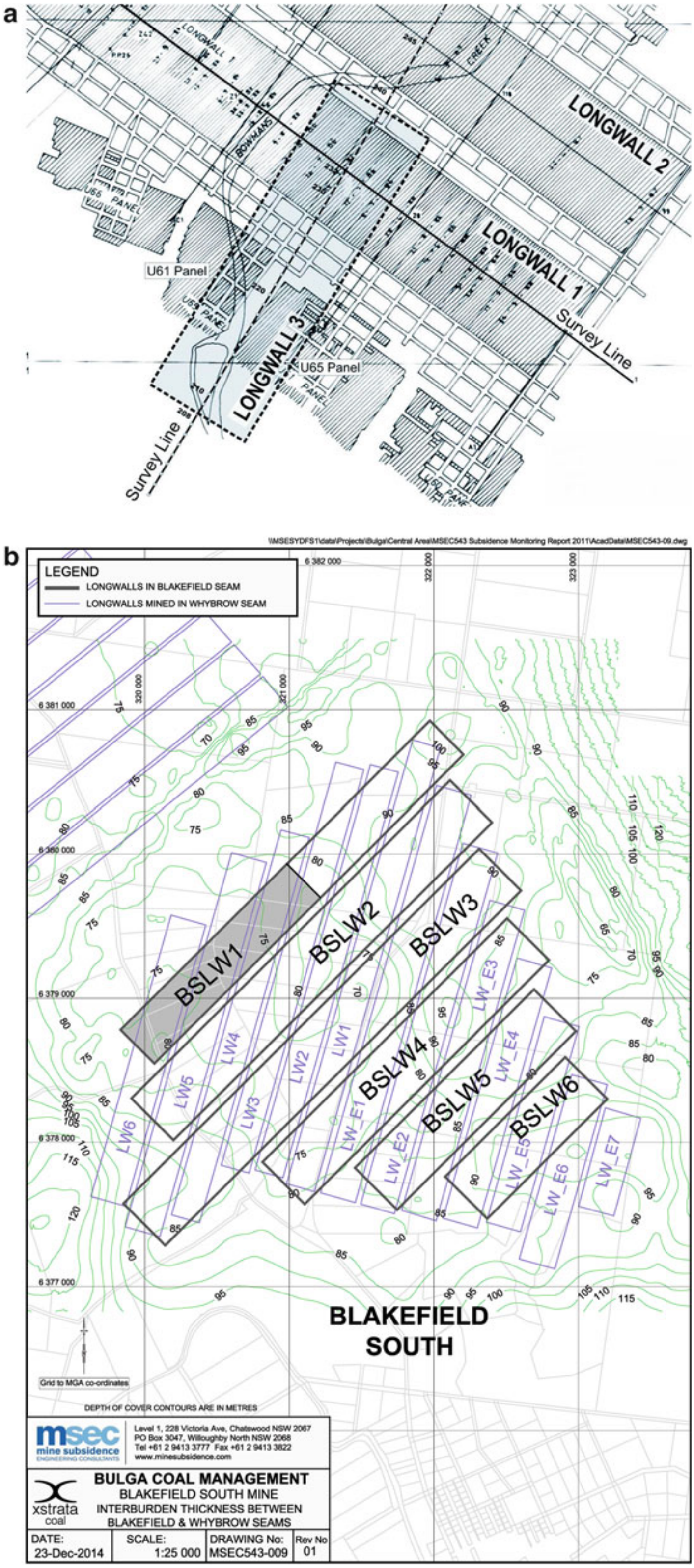
As is apparent from Fig. 5.21, superimposing the gateroads exposes the lower seam gateroads to elevated abutment stresses. The practice is mostly confined to single entry situations. Experience indicates that indented roadways need to be located at least 10 m inside the upper seam workings, even when the interburden thickness is as low as 1 m, thus resulting in a substantial reduction in panel width (Galvin 1997b). Gateroad layouts which locate the cut-throughs within the interpanel pillars result in these roadways being exposed to high abutment stress and to the surface being subjected to high strains, changes in gradient and cyclic depressions. Driving new sets of gateroads totally under the goaves of the overlying seam places the cut-throughs in destressed conditions and reduces strains and gradients at the surface but results in a very significant reduction in longwall panel width (Fig. 5.21b), and does not eliminate cyclic depressions in the surface profile.

Offsetting the gateroads overcomes most of the previously noted limitations but introduces the risk of longwall face control being adversely impacted by interaction with the overlying chain pillars. Offsetting places gateroads in destressed conditions, provides total flexibility in selecting longwall panel width, and minimises surface strains, tilts and cyclic depressions in ground profile. Its success depends on the appropriate selection of longwall powered supports and the robustness of the Ground Control Management Plan for maintaining stability in the zone of elevated stress associated with the overlying interpanel pillars. Remnant pillars often have particularly adverse implications for floor control as well as for roof and face control, with loss of floor bearing capacity having sometimes resulted in uplift and tilting of the AFC, bending of relays bars, jamming of the shearer between the floor and the canopies of the powered supports, and bogging and rotation of the powered supports.

In practice, a range of factors such as dip, water management and gas management, old workings, geological features and pre-existing mining-induced fracturing often prevent multiseam workings from being orientated in the same direction in each seam. Hence, in the case of multiseam longwall workings, it may be inevitable that gateroads are not only offset but also that extraction direction fluctuates between working from goaf to solid and from solid to goaf during the course of extracting a panel. Two examples are shown in Fig. 5.22. This can offer the indirect benefit of the working face of the second seam operations not being subjected simultaneously to high abutment stress along it full length as it passes beneath or over the abutment of an earlier extracted seam.

Fracturing associated with caving and subsidence enhances the potential for fluid flow (mine ventilation, water, gas etc.) through the goaf. This potential is higher in multiseam situations because the greater overall extracted seam thickness results in more extensive and intense strata deformation and because the goaf may not have had sufficient time to reconsolidate. Significant volumes of water stored in fracture networks in dilated and constrained zones (Fig. 3.5) may also be released when subsidence is reactivated. Permanent flexure in the transition zone between subsided excavations and panel abutments is another important consideration. There is a history of flexure zones providing a conduit for fluid flow into lower seam workings from overlying workings and the surface. It should be borne in mind that fluid in a lower seam may be under a pressure head and, therefore, also present an inundation or inrush hazard in multiseam mining situations. Fracturing, in general, creates the potential for ventilation cross-connections between the various seams and, therefore, can have serious implications for managing gas and spontaneous combustion.

A range of factors determine the structural integrity and load bearing capacity of the interburden and how these attributes impact on interaction between multiseam workings. The more obvious and influential are material strength properties and natural and mining-induced fracturing. Stratification is also a significant parameter, with numerical modelling by Gale (2003) and Lightfoot and Liu (2010), for example, giving insight into how shear stresses on bedding planes and pre-existing fractures act to significantly destabilise a roadway. The


a
LONGWALL 2
LONGWALL 1
LONGWALL 3
Survey Line
Survey Line
U61 Panel
U65 Panel
BOWMANS
CREEK
b
\\MSESYDFS1\data\Projects\Bulga\Central Area\MSEC543 Subsidence Monitoring Report 2011\AcadData\MSEC543-09.dwg
LEGEND
LONGWALLS IN BLAKEFIELD SEAM
LONGWALLS MINED IN WHYBROW SEAM
BSLW1
BSLW2
BSLW3
BSLW4
BSLW5
BSLW6
LW1
LW2
LW3
LW4
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LW6
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BLAKEFIELD SOUTH
Grid to MGA co-ordinates
DEPTH OF COVER CONTOURS ARE IN METRES
msec mine subsidence ENGINEERING CONSULTANTS
Level 1, 228 Victoria Ave, Chatswood NSW 2067
PO Box 3047, Willoughby North NSW 2068
Tel +61 2 9413 3777 Fax +61 2 9413 3822
www.minesubsidence.com
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BULGA COAL MANAGEMENT
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INTERBURDEN THICKNESS BETWEEN
BLAKEFIELD & WHYBROW SEAMS
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**Fig. 5.22** Examples of multiseam longwall panels in the Hunter Valley of NSW that are orientated in different directions between seams. (**a**) Multiseam layout evolved over time (After Li et al. 2010). (**b**) Purpose designed multiseam layout

modelling demonstrates the need to consider all components of stress and strain in complex multiseam geometries.

Many of the preceding principles have been developed on the basis that mining occurs in descending order. When mining occurs in ascending order, careful consideration also needs to be given to pre-existing caving and fracturing in the upper seam mining horizon.

Given that multiseam mining can be impacted by remnant pillars in neighbouring seams, flexure zones and the direction of mining over and under existing goaf edges, it is important that accurate records are maintained of actual ('as-built') extraction and that goaf edges and remnant pillars in adjacent seams are recorded on Hazard Plans (see Sect. 12.6.1). The need to maintain accurate extraction records is particularly important in pillar extraction where there is both a higher propensity to leave remnant pillars and for these not to conform to a regular layout.

#### 5.3.3.1 Design Considerations

Numerous permutations of mining method, layout and extraction sequence can be associated with multiseam mining. The primary factors that need to be taken into account are generally the same and include:

- direction of natural and mining-induced fracturing;
- stress magnitude and distribution;
- strain magnitude and distribution;
- interburden thickness;
- interburden stratification;
- interburden strength;
- time interval between extracting seams;
- floor heave;
- panel orientation relative to panels in adjacent seams;
- flexure zones;
- gas and water flow;
- surface subsidence; and
- economic face lengths and development to extraction ratios.

In the case of multiseam bord and pillar workings, the sequence of seam extraction is not usually critical. However, there may be benefits in extracting the least competent seam last so that it remains in a confined state when subjected to displacement associated with extraction of the other seams.

The total extraction of an upper seam or the top portion of a thick seam prior to extracting deeper coal offers the potential to alleviate pressure bursts and gas outbursts and is a common practice in some tabular metalliferous mines and in coal mining in Europe. Palarski (1999) reports that in Poland, it is often necessary to destress the rock mass and mitigate the risk of pressure burst by extracting the upper slice of a thick seam and then backfilling subsequent slices.

In many coal mining countries, maximum vertical displacement at the surface arising from the extraction of a single seam is typically of the order of 50–65 % of the extracted height. However, extraction of subsequent seams results in proportionally greater subsidence, variously reported to be of the order of 90–100 % of the incremental extracted height (see for example, Galvin 1981; Schumann 1993; Lear and Schumann 1989; Van der Merwe 1989; Li et al. 2010). It has been conjectured that this is due to either or both enhanced caving of the immediate roof of the second seam and reconsolidation of the goaf of the first seam workings.

Historically, the design of multi-seam workings has been based on a range of empirical relationships, supplemented in some cases by basic analytical and numerical analyses. Overviews of many of these approaches have been provided by Haycocks and Zhou (1990), Hsiung and Peng (1987), Mark (2007) and Peng (2008). The techniques are generally confined to simple and regular mining layouts and geometries. Even then, care is still required, with many of the more recognised guidelines subsequently having been found to be incomplete, erroneous or applicable to only a narrow range of circumstances. Galvin and Anderson

(1986) and Galvin (1988) detailed the limitations associated with the Wardell Guidelines (Wardell 1975) for multiseam workings, which have found extensive application in Australia. Bradbury and Hill (1989) and Hill (1989) have identified some of the constraints associated with the criteria of Salamon and Oravecz (1976) for multiseam first workings which have found extensive international application.

Often, mines wishing to undertake multiseam operations were not planned from the outset with this in mind, as in the case shown in Fig. 5.22a. In these cases, the multiseam mining layouts are generally complex in three dimensions and require consideration of many different permutations of parameters that can influence success. Similar situations are also often unavoidable in purpose designed multiseam mines (Fig. 5.22b). As recognised by Mark (2007), these situations defy empirical estimation.

Mark (2007) approached this problem by applying logistic regression to a US database comprising 344 case histories sourced from 44 mines to develop a model that best accounted for required interburden thickness based on successful and unsuccessful experiences of multiseam mining. Because this model was derived from logistic regression, it cannot be assumed to be mechanistically valid. The unsuccessful cases included those where mining was completed, but with significant difficulties attributed to multiseam interactions concluded from evidence on mine plans of roof falls, roadways that were not fully developed, and pillars that were left unmined. In practice, particularly where these events are pre-empted and appropriate controls implemented, some operators may still consider mining to have been successful.

Since early 2000, significant advances have been made in the numerical modelling of multiseam workings, as illustrated by reference to Gale (2003), Munsamy et al. (2004), Zipf (2005) and Lightfoot and Liu (2010). These go a considerable way to addressing the limitations associated with empirical approaches to mine design. Developments in inelastic models offer the opportunity to simulate aspects such as fracture development and stress path so as to more accurately understand and predict the extent and severity of pre-existing strata disturbance at targeted mining horizons. This is particularly important for predicting the location and nature of highly stressed ground, fractured and destressed ground, and flexure zones, in order to inform the selection of the appropriate mine layouts, mining systems, support design philosophies and support and reinforcement technologies and techniques.

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# Support and Reinforcement Systems

# 6

## Abstract

Once an excavation is formed, it must be ventilated and made secure before persons can venture through it. A wide range of ground support and reinforcement systems are available for securing the surfaces of underground excavations in coal mines. This chapter is focussed on identifying these systems and providing a mechanistic understanding of how they function. This provides an engineering basis for selecting suitable support and reinforcement systems, installing them in an effective manner, and appropriately monitoring the resulting ground response.

The chapter commences by identifying the primary characteristics of any ground support system, being initial stiffness, load capacity, yield capacity and, where appropriate, stability. The distinction is made between the function of a support element and that of a reinforcing element. It then goes on to evaluate support and reinforcement systems under the headings of standing support; tendon support and reinforcing elements; surface restraint systems; spiling; strata binders; and void fillers.

A considerable portion of the chapter is devoted to the anchorage methods for tendon support systems as these play a critical role in tendon performance. The principles of classical beam theory presented in Chap. 2 are then invoked and developed further to provide direction as the type, location, density and timing of installation of ground support systems. The chapter is supported with an extensive selection of photographic illustrations of these systems installed in coal mines along with other figures to help the reader visualise and better understand the underpinning engineering principles.

## Keywords

Anchorage • Angled bolt • Beam building • Bedding shear • Bending stress • Cable bolt • Chock • Crib • Cross support • Flexural strength • Gloving • Ground reinforcement • Ground response curve • Ground support • Grout anchor • Load transfer • Mechanical anchor • Parting plane • Polyurethane resin • Post-grouting • Pretensioning • Primary support • Prop • Resin anchor • Rock bolt • Safety precautions •

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Screen • Secondary support • Shotcrete • Spiling • Standing support • Steel arch • Strata binder • Stress corrosion cracking • Support density • Support resistance • Suspension • Thin spray-on liner • Thrust bolting • Truss • Void filler • Yield strength • Yielding support

## 6.1 Introduction

The term 'support' is used generically to describe natural and artificial systems employed to limit rock mass displacement. However, when considering the mechanics of how these systems actually control and modify rock mass response, it is helpful to distinguish between elements that contribute to 'supporting' the rock mass and elements that contribute to 'reinforcing' it. Similar to Brady and Brown (2006), this text adopts the definitions introduced by Windsor and Thompson (1993), being:

- **Support** is the application of a reactive force to the surface of an excavation and includes techniques and devices such as timber, fill, shotcrete, mesh, and steel or concrete sets or liners.
- **Reinforcement** is a means of conserving or improving the overall rock mass properties from within the rock mass by techniques such as rock bolts, cable bolts and ground anchors.

Both support and reinforcement measures may be **passive**, in which case displacement of the rock mass is required to generate a reactive force to resist that movement, or **active**, in which case the support element also applies a pre-load to the rock mass. In general, support is utilised primarily to resist displacement while reinforcement is intended to strengthen the rock mass to prevent or severely restrict displacement, but with a capacity to also control excessive displacement.

**Primary support** describes all support and reinforcement measures applied during or immediately after excavation, to ensure safe working conditions during subsequent excavation, and to initiate the process of mobilising and conserving rock mass strength by controlling boundary displacements (Brady and Brown 2006). In underground coal mining, **secondary support** refers to additional support and reinforcement installed some distance back from the face and/or some time after excavation in anticipation of a changed loading environment or in response to deteriorating ground conditions. Additional support and reinforcement required subsequent to the installation of secondary support is referred to as **tertiary support**.

In this text, the term **ground support** refers generally to any measure for controlling ground movement, irrespective of whether, from a theoretical perspective, it performs a support function or a reinforcement function. Custom and practice, colloquialism and jargon give rise to some exceptions which the reader should be able to distinguish. Mobile roof supports (MRS) and longwall powered supports are discussed in the chapters on pillar extraction and longwall mining, respectively.

## 6.2 Primary Characteristics

Table 6.1 lists and defines a number of terms that find common usage when discussing capacity aspects of ground support systems. The term 'support' is used in the generic sense in these definitions and may include reinforcement systems.

The primary characteristics of a ground support system are initial stiffness, load capacity, yield capacity and, in the case of a roof to floor system, stability. The concept of a ground response curve, introduced in Sect. 2.6.12 and shown in Fig. 6.1, illustrates some of these attributes. The intersection of a support response curve with the ground response curve defines the point where equilibrium is reached and no further convergence takes place unless external changes occur. A high initial stiffness is usually a desirable property of a support in order that it rapidly generates resistance to displacement. This

**Table 6.1** Terms related to support capacity and performance and their definition

| Term | Unit | Definition |
|---|---|---|
| Support load | N | The deadweight load acting on a support or reinforcement element |
| Support force | N | The force generated in a support or reinforcement element and on the rock mass by a support or reinforcement element. The force is a function of the deformation properties of the support or reinforcement element because the loading system is displacement controlled |
| Support density | Number of support units/m$^2$ | The number of support or reinforcement elements per square metre of rock surface |
| Setting load | N | The force generated in a support or reinforcement element by the installation process |
| Yield load | N | In respect of hydraulic and friction support systems, is the maximum load that the support will carry before discharging hydraulic fluid or otherwise slipping so as to converge, or yield, in order to avoid accepting further load |
| | | In respect of most other types of support and reinforcement elements, it is the load corresponding with the onset of permanent deformation or non-recoverable displacement in the support or reinforcement element |
| Support resistance | N/m$^2$ | Maximum support or reinforcement force generated per unit area. It equals maximum support force per support or reinforcement element x support density |

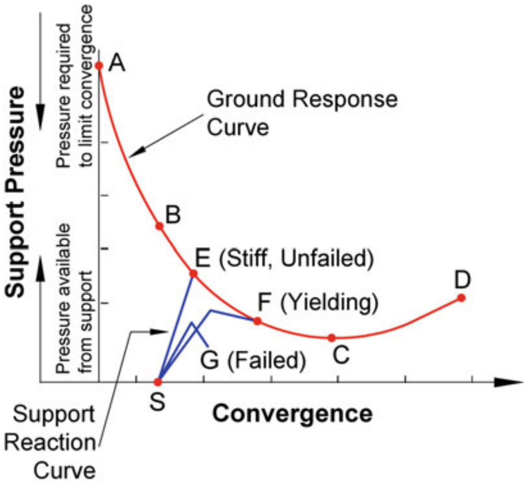

**Fig. 6.1** A conceptual ground response curve showing some of the possible interactions with support systems (Adapted from Daemen 1977)

corresponds to the section labelled SE in Fig. 6.1. Should the peak capacity of the ground support system be exceeded, a capability to yield over an extended range is helpful in maintaining a high residual support resistance to confine the fractured rock mass and in promoting load transfer further into the rock mass. This may even result in equilibrium being re-established, as shown by section SF. The support response curve labelled SG defines a situation where equilibrium is not reached and convergence continues. The earlier the ground support is installed, the more effective it will be, provided that it has adequate support capacity to reach equilibrium with the rock mass.

## 6.3 Standing Support

**Standing support** refers to systems installed between the excavation roof and floor to restrict convergence. These principally comprise wooden and hydraulic props, timber chocks, cementitious columns and rock pillars which are loaded by floor and roof displacement. Most artificial standing support systems offer the benefits of being able to be installed quickly, in confined spaces, and without the need for services (water, compressed air, electricity).

From an operational perspective, standing supports are often undesirable because they can impede ventilation flow, personnel and materials transport equipment, and access. However, these disadvantages are often outweighed by ground control advantages in circumstances where standing support provides the only opportunity to install secondary support in time to prevent a roof fall. This can be particularly important in situations where machinery cannot gain access to recover a fall of ground and where there are limited options for stowing fallen material underground or removing it from the mine. Standing

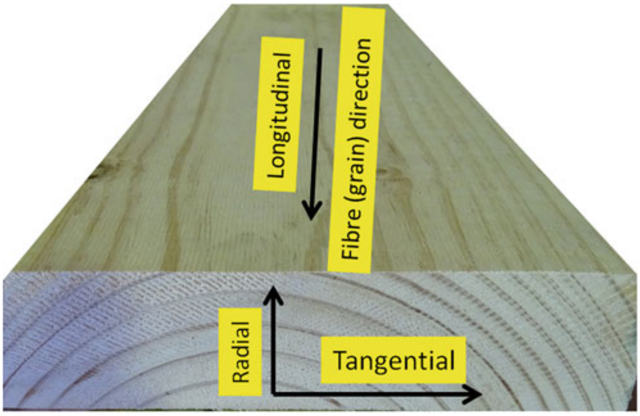

**Fig. 6.2** Illustration of the orthotropic nature of wood

support also provides one of the most effective means of controlling floor heave.

Timber still forms the basis of many standing support systems, although its use is rapidly declining. The strength and yielding properties of timber supports are influenced principally by the direction of loading relative to the grain of the timber; timber moisture content; timber density; and the presence of growth features. Wood is an orthotropic material, exhibiting independent properties in three mutually perpendicular directions (Fig. 6.2). In compression, it is strongest when loaded radially and weakest when loaded tangentially. These characteristics are reversed in tension and can be utilised to advantage in timber support systems.

Timber strength increases as the free water content in the cell spaces decreases. During the process of seasoning, the air-dry moisture content of so-called ‘green’ timber reduces from around 30 % to 10 to 15 %, resulting in shrinkage. Both Australian and USA standards specify that seasoned timber strengths be quoted at 12 % moisture. Timber is weaker when green but is much easier and quicker to cut. It is usual, therefore, for mine timber that needs to be cut to size on the job to be supplied in a green state. In some situations, this may require support systems constructed from timber to be retightened periodically as the timber dries and shrinks.

Knots are a common growth feature and are generally considered to have little effect on the compressive strength of timber but may seriously decrease bending strength. These parameters, together with hardness index, are taken into account when assigning a structural grade to timber. An overview by Barczak (2005) of standing support practices in the USA provides further information on the attributes of timber-based support systems.

### 6.3.1 Props

Timber props offer the benefits of being relatively light, stiff, and providing visual and audible warning of displacement, although the latter should not be relied upon. Bark should be removed to enhance observation of prop behaviour. The main support related disadvantages with timber props are a lack of yield capacity and poor lateral stability when impacted by goaf falls or by ride in dipping seams.

The performance of a timber prop is a function of timber species, roof and floor competence, shape of prop ends, presence and type of prop end caps, installation technique and prop slenderness ratio. The compression of a prop at maximum load varies with timber species but is typically around 1 %. A prop is capable of generating high point loads and **end caps** (also referred to generally as **lids** or **boards** or specifically as **headboards** and **footboards**) may be required to prevent punching of soft or weak strata. End caps can also impart a yielding capacity to a prop, as does chamfering or tapering (pointing) one end of the prop to reduce prop stiffness. Chamfering should be restricted to reducing the prop cross-sectional area by no more than 50 %.

Australian underground coal mining experience has shown that as a general rule, the diameter of a eucalypt hardwood prop should be 25 mm (one inch) for every 300 mm (one foot) of prop length (metric approximated from Menzies c1970). This generates the metric prop dimensions given in Appendix 6. Test results confirm that the convergence profile and failure load of a prop are both affected significantly by the nature of timber end caps, pointed prop ends, and tightening wedges. For example, the load capacity of a 2.45 m long, 175 mm diameter eucalypt prop without pointed ends is typically of the order of 500–600 kN (50–60 t), with convergence at failure ranging from around 20 mm in the absence of end caps and tightening wedges, to around 80 mm when set with a 100 mm thick softwood cap. Pointed props of similar dimension have a failure load in the range of 250–350 kN (25–35 t), with convergence at failure ranging between 125 and 350 mm, depending on the nature of the point profile (metric approximated from Menzies c1970). A range of mine prop strengths and dimensions for select Australian, South African and USA timber species and circumstances is presented in Appendix 6. These demonstrate how prop strength rapidly reduces with increasing length.

There is a practical limit to the size of a prop that can be utilised in high mine workings without compromising support capacity. In order to maintain the same resistance to buckling, the diameter of a prop has to increase in direct proportion to the square root of its length. $l_p$. Hence a doubling of mining height requires prop diameter, $d_p$, to increase by a factor of √2, or approximately 1.41. Since the mass of the prop is proportional to $l_p d_p^2$, it follows that a doubling of mining height increases the mass of the prop by a factor of four. This illustrates the practical limitations of using timber props of sizeable diameter in high workings, as apparent in Figs. 6.3 and 6.4.

When developing support designs and safe working procedures, it is important to make allowance for the effect of field conditions on prop performance. Prop strength may be reduced as a result of eccentric loading because it is not always possible or practical to set a prop so that its ends are parallel to the roof or floor. Load is unlikely to be shared equally amongst all props in a row due to differences in factors such as prop setting procedure, preload, diameter and length. Both floor and roof convergence can be variable along a row of props. If a seam is dipping, props should be set at an inclination that is midway between vertical and normal to the seam floor.

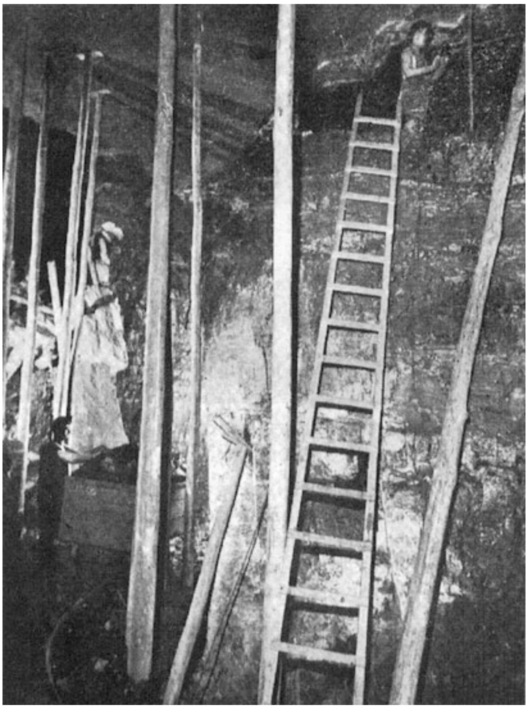

**Fig. 6.3** High workings at Abermain Colliery, Australia – note mine worker at the top of the ladder (After Martin et al. 1993)

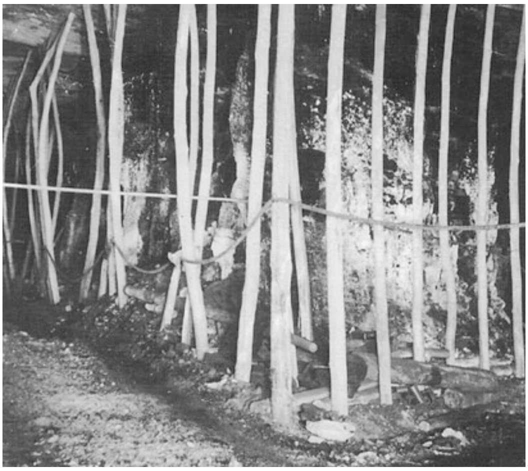

**Fig. 6.4** High workings at Bellbird Colliery, Australia, illustrating the propensity for long slender props to buckle (After Martin et al. 1993)

When timber support has to remain functional over extended periods of time, particularly in hot and humid environments, the timber should be impregnated to prevent rotting, decay and attack by pests.

Mechanisation and new support technologies have resulted in a significant reduction in the use of timber props in many coal mining operations, to the point where training is no longer offered at some mine sites in how to set a prop. Some mine workers can go for years without having to set a prop and, hence, the art of setting a prop is in danger of being lost. There are still many circumstances where timber props may need to be set. These include to support the lip of a fall; to support cross supports; to provide blanket temporary support to deteriorating roof while other slower ground support techniques are implemented; to supplement mobile roof supports in pillar extraction; to act as breaker props in longwall face recovery operations; and to reach equipment immobilised under unsupported roof. Hence, Appendix 7 details a safe work procedure for setting a timber prop. As with all such procedures, it should be risk assessed against site-specific conditions and modified as required in order to achieve safe and effective outcomes.

Hydraulic props offer a number of advantages over timber props in many situations because they are quicker to set; can be set to their maximum load capacity at the time of installation; the quality of their installation is largely independent of the operators; their capacity is known and consistent; they can incorporate a high and rapid yield capacity; they offer a flexible range in working height; and they are reusable. However, hydraulic props can be heavy and unwieldy to handle in moderate to high workings; may expose operators to a risk of fall of ground when being withdrawn; and are expensive if conditions do not permit them to be reclaimed.

Friction props comprise inner and outer tubes, typically 50–75 mm in diameter, that rely on some form of steel-on-steel sliding mechanism to set the prop and, in some cases, to impart a capacity to yield. Since the dynamic coefficient of friction is less than the static coefficient of friction, stick-slip behaviour can be experienced with these types of props, resulting in a complete breakdown in support resistance in dynamic loading conditions.

A range of specialised props provide for either or both a high support capacity and an extended yielding capacity. These comprise, for example, timber elements machined to size so that they can slide within a steel pipe and clusters of props with split ends. Load capacity of some systems can be as high as 1.4–1.7 MN (140–170 t) at 50 mm displacement provided that they are prestressed, while others can accommodate convergence of 30–40 % of prop length prior to failure. A disadvantage of some of these products is their considerable weight. Examples of the stiffness and load capacity of a system comprising a (proprietary brand) cluster of props and how these properties compare to CAN® monolithic cementitious columns are shown in Fig. 6.5.

In the past, timber props have often been recommended as a temporary support measure to protect operators while they are installing more permanent forms of support. This presents a conundrum, especially when the roof is in an unsupported state, as the action of setting a prop as temporary support can expose the operators to the very risks that the prop is intended to protect against. Not only do operators have to work under unsupported roof while setting the prop but the actions involved in this process, especially hammering, are conducive to initiating a fall of ground. Serious and fatal accidents have resulted, with legal proceedings highlighting the illogical nature of this concept. The potential to raise and lower a hydraulic prop by remote means has been exploited in a number of temporary support concepts which address safety concerns associated with using timber props under unsupported or deformed roof. Figure 6.6 shows an example of an award winning device that permits hydraulic props to be set remotely.

### 6.3.2 Timber Chocks

Timber chocks (also referred to as cribs, pigsties and packs) can take a variety of forms, some of which are shown in Fig. 6.7. Sawn timber

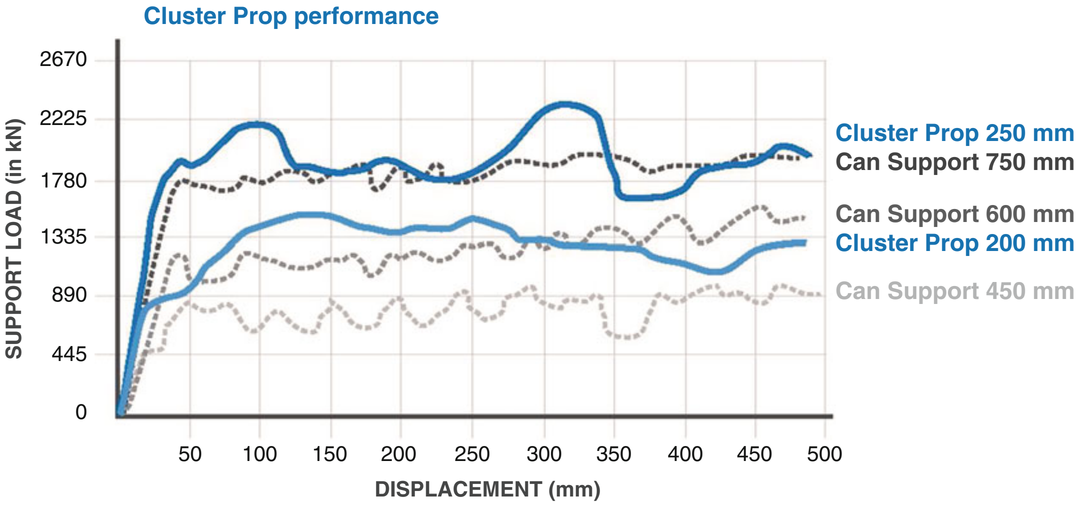

**Fig. 6.5** An example of the performance characteristics of a specialised timber prop system comprising a (proprietary brand) cluster of three props and how these compare to various CAN® cementitious monolithic products

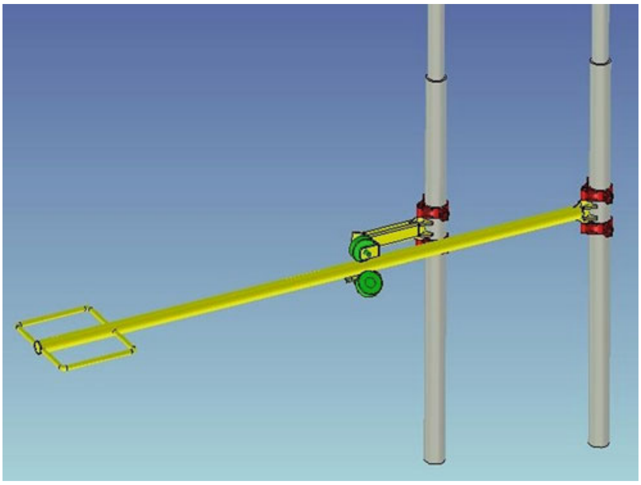

**Fig. 6.6** An innovation for advancing and setting props by remote means

elements of rectangular cross-section typically range from 25 to 150 mm in thickness, 100–200 mm in width and from 0.9 to 2.1 m in length. Practical considerations, in particular manual handling risks, speed of construction, and maintaining an access way past chocks, now result in most chocks being constructed from sawn hardwood slabs that are nominally 100 mm × 100 mm or 100 mm × 150 mm in cross section and 1.2–1.5 m in length. Because they can be constructed on site from short lengths of timber, timber chocks are suited to being installed in areas that have limited access and services.

Chock performance varies widely with timber species, the section of the log from which elements are sawn and the direction in which these elements are loaded. The presence of heartwood, which Australian Standards define to be the centre 100 mm of a log, makes loading in the weakest direction parallel to growth rings unavoidable and causes the timber to be prone to severe cracking upon drying, as shown in Fig. 6.8. Under Australian Standard AS 1720

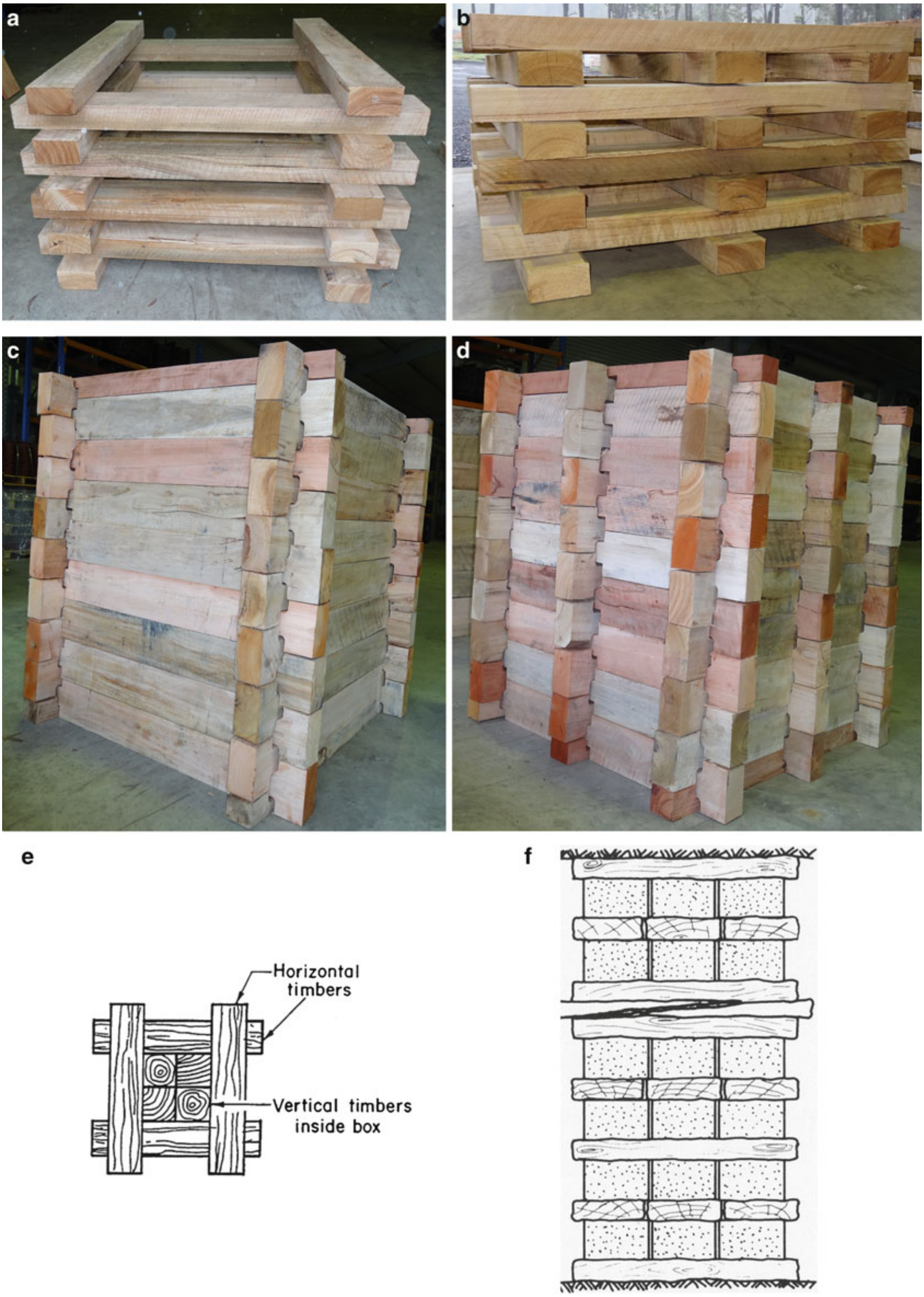
a
b
c
d
e
Horizontal timbers
Vertical timbers inside box
f

**Fig. 6.7** Examples of different types of timber-based chock constructions. (**a**) 4 point chock, (**b**) 9 point chock, (**c**) 4 point Link-n-Lock® chock, (**d**) 9 point Link-n-Lock® chock, (**e**) Confined Core Crib (After Barczak and Gearhart 1994), (**f**) Sandwich chock (After Cook et al. 1974)

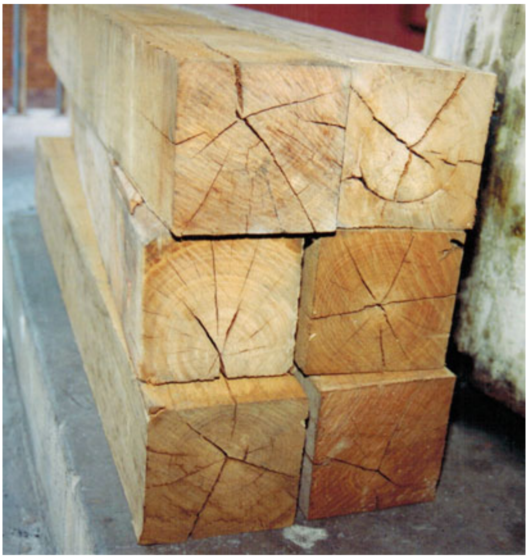

**Fig. 6.8** Chock timber elements adversely affected by having been cut from heartwood (After Galvin et al. 1996)

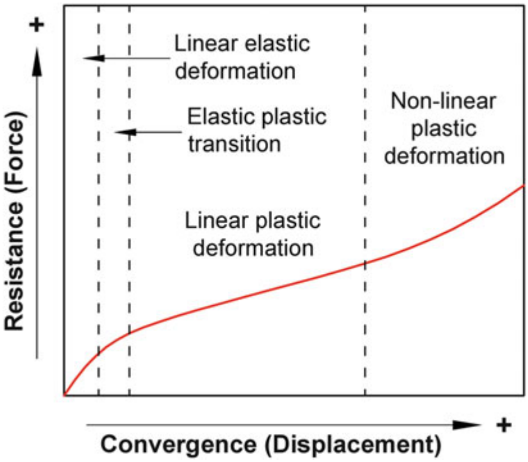

**Fig. 6.9** Generalised load-convergence relationship for wooden crib supports (Adapted from Barczak and Gearhart 1993)

relating to timber properties and grading, the chock timber in Fig. 6.8 would not qualify as structural grade but rather as landscape grade intended for non-load bearing applications. Single point chock constructions made of this timber disintegrated at one quarter of the load and convergence of similar chocks constructed from structural grade timber (Galvin et al. 1996).

The most important chock performance parameters from an operational perspective are:

- initial chock stiffness;
- ultimate chock strength;
- ultimate chock displacement; and
- chock stability during convergence.

Figure 6.9 depicts a generalised load-convergence curve for a hardwood chock. Barczak and Gearhart (1993) described how during the initial loading stage which extends up to around 5 % strain, the resistive force of the timber chock increases quickly as a linear function of convergence in what is termed the 'linear elastic' deformation phase. The chock stiffness then decreases as the timber goes through an 'elastic-plastic' transition phase over about a 5 % strain range and into a 'linear plastic' deformation stage, which may continue up to 25–30 % strain. The chock stiffness then increases non-linearly in what is identified as the 'non-linear plastic' deformation phase. During this last stage, the cellular structure of the wood collapses and the ultimate strength of the chock is realised. This behaviour is reflected in full scale testing outcomes for 4 point chocks constructed from eucalypt, presented in Fig. 6.10.

When the intention of installing chocks is to prevent roof failure, the stiffness of the chocks in the linear plastic phase is critical. If deformation has already commenced, a high initial stiffness still offers advantages by providing greater confinement to the roof and floor to control the rate of convergence and roof failure. Due to the fibrous and cohesive nature of timber, it remains in a confined state under the effects of friction and continues to accept load as it passes through the linear plastic and non-linear plastic deformation phases. If the stability of the overall chock structure can be maintained, the chock may continue to accept load indefinitely.

Chock stability increases with increase in aspect ratio (or width-to-height ratio) and moment of inertia. Barczak and Gearhart (1993) suggest for USA operations that aspect ratios of less than 0.23 threaten chock stability, while ratios greater than 0.4 are not cost effective. The aspect ratio in Australian operations ranges

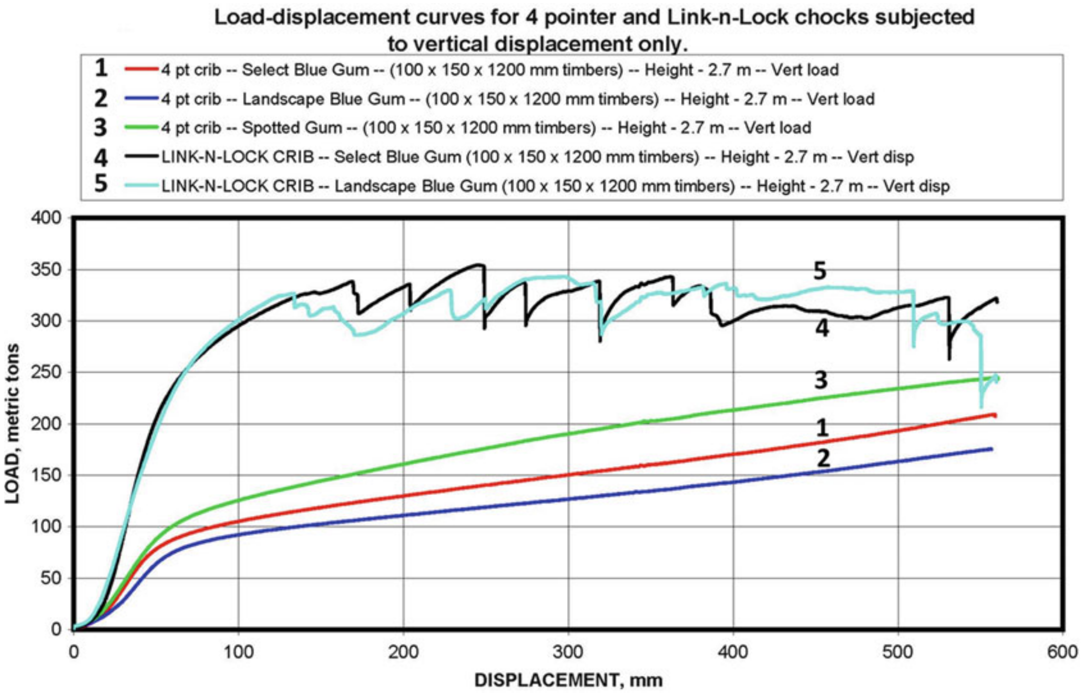

**Fig. 6.10** Full scale testing outcomes on 4 point and Link-n-Lock® timber chocks (After Offner et al. 1999)

from around 0.3–0.7, with aspect ratios greater than 0.4 being cost effective when the immediate roof is very weak and friable because chock density is minimised by spreading the load over a greater area (Galvin et al. 1996).

Common sources of eccentric loading of chocks to be avoided are:

- out-of-plumb construction;
- poor foundation preparation and/or foundation failure;
- non-uniform setting (preloading) of the chock;
- insufficient timber overhang between chock layers, resulting in premature shear failure of the timber elements, such as that illustrated in Fig. 6.11, because the timber ends are not constrained by the effect of punching into each other;
- mixed species of timber in the chock construction; and
- a mix of radial and tangential loading of timber elements.

In respect of four point chocks, studies by Galvin et al. (1996) concluded that:

- Australian Standards for timber provide a good basis for predicting the relative performance of different species and structural grades of chock timber elements.
- Timber species should not be mixed in a chock construction.
- Control over saw-milling and construction practices to maximise the extent that growth rings can be orientated near parallel to the load-bearing face of a timber element can significantly improve chock performance at no additional cost to the operator.
- All timber elements containing heartwood or extensive shrinkage cracks should be rejected.
- Chocks should have an aspect ratio of at least 0.25.
- Timbers should be placed such that they overhang by at least 50 % of their width.
- If chock constructions are not loaded soon after construction, they should be periodically

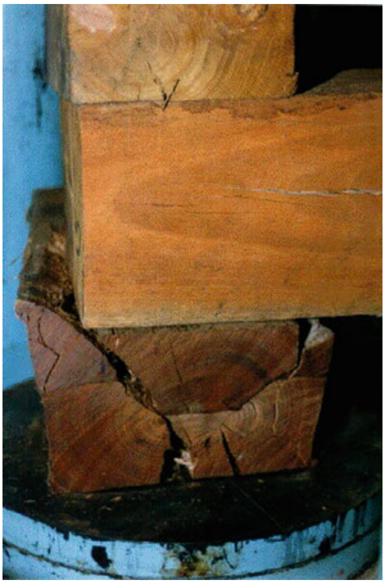

**Fig. 6.11** Shear failure of the end of a chock timber due to insufficient overhang between timber layers (After Offner et al. 1999)

retightened to take up any shrinkage caused by the timber drying out.

A range of alternative chock constructions have been developed to address shortcomings in point chock constructions and to make more cost-effective use of timber. The Link-n-Lock® form of construction, illustrated in Fig. 6.7c, d, is designed to increase the timber contact area between layers from the 15 to 30 % typically associated with four and six pointer chocks, respectively, to 100 %. A comparison between the load displacement characteristics of the two types of chocks measured in full-scale laboratory testing is shown in Fig. 6.10. It must be borne in mind that the absolute values associated with the test results are influenced by moisture content of the timber at the time of testing and the rate that load is applied to the chocks, which is many times greater than that in most underground situations.

Offner et al. (1999) concluded that in comparison to conventional 4 point chocks of the same timber species and similar dimension, the Link-n-Lock® construction resulted in:

- a doubling in stiffness in the linear elastic range;
- a trebling in load capacity in the linear elastic range;
- a steady state yield characteristic in the linear plastic range;
- a halving in the cost per 10 kN (1 t) of support force generated;
- quality control in placing timber in the optimum orientation with respect to its grain, because this can be engineered into the manufacture of the timber elements rather than being left to the judgement of the mine workers; and
- improved chock stability during construction and under load.

In coal mining, timber chocks find most application as a yielding support system in longwall tailgate roadways, being able to generate a support force of several MN (hundreds of tonnes) at strains of over 20 %. However, these types of system are relatively soft, requiring some 50–100 mm of convergence to generate substantial support resistance. Therefore, it is important to prestress chocks and to maintain them in a stressed state, especially if the timber was not seasoned at the time of construction.

### 6.3.3 Cementitious Chocks

Most cementitious-based chocks are comprised of either a stack of concrete-based elements or a monolithic column. Timber elements may be intermixed with concrete elements to impart yielding characteristics, although extrusion of the timber under load is prone to induce lateral tension in the concrete elements and result in premature failure of the chock. Hence, the market for cementitious based chocks in underground coal mining is dominated by two types of yieldable monolithic chock, one type that is precast offsite in a large diameter, thin wall, metal tube, or ‘can’, and the other constructed by pumping cementitious grout into a fabric bag

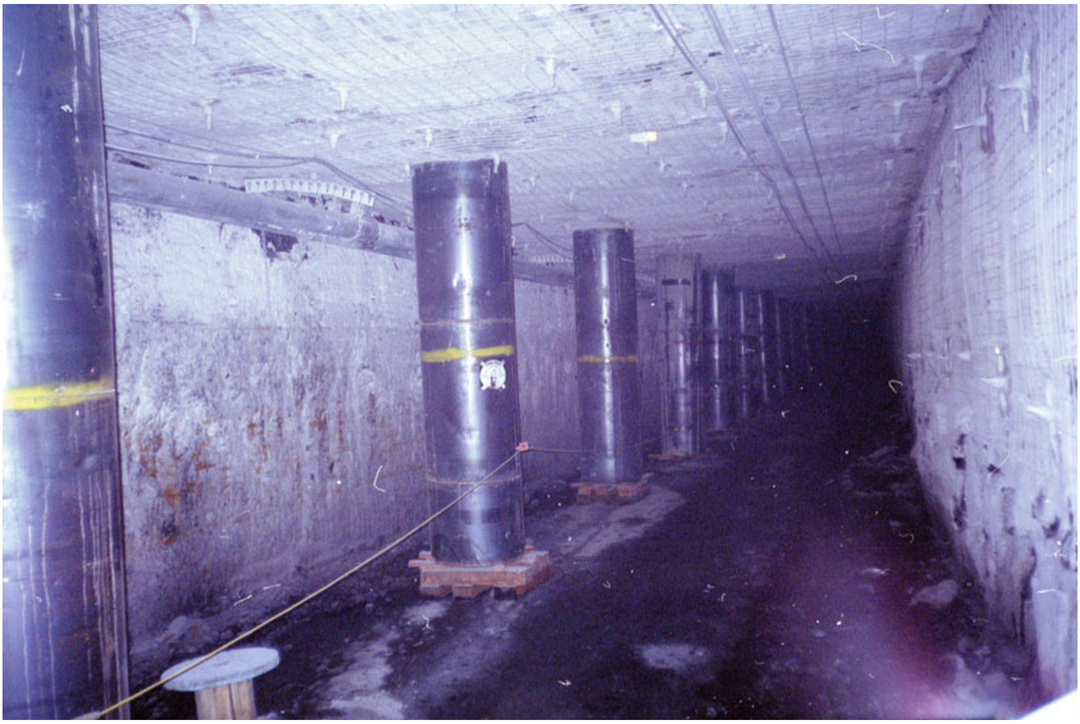

**Fig. 6.12** 700 mm diameter CAN® proprietary brand monolithic cementitious chocks installed in a longwall tailgate

suspended from the mine roof at the installation site. The former is generally referred to as a CAN® and the latter as a 'pumpable crib'. Figure 6.12 shows a series of CAN® monolithic cementitious chocks installed in a longwall tailgate.

The load-displacement performance parameters already noted for timber chocks also apply to cementitious based chocks. Figure 6.13 shows the load-displacement characteristics determined in full scale laboratory testing for a 560 mm (22 inch) diameter CAN® and a 760 mm (30 inch) diameter pumpable crib, both 1.8 m high. Field monitoring confirmed that the in situ yield load of a CAN® was close to that measured in the laboratory. Provided that the chocks have been prestressed, both structures have a similar initial stiffness, with yield and peak load being achieved after about 25 mm of convergence. A comparison between Figs. 6.10 and 6.13 shows that this stiffness is high in comparison to that of timber point chocks. The yield behaviour of CANs® and Link-n-Lock® chocks is not dissimilar.

The cross-sectional area of cementitious based chocks is small, both relative to their load carrying capacity and in absolute terms. This makes them prone to punching soft or weak roof and floor strata, unless fitted with end plates to spread the load (see for example, Fig. 9.26d). It also increases the opportunity for the immediate roof to unravel between chocks if the chocks are not closely spaced or used in conjunction with some form of surface support system (such as straps or mesh). Support stability can also become problematic when chock aspect ratio drops below 0.2 (Dolinar 2010). More detailed discussion of these and other performance aspects in relation to CAN® supports is provided by Gearhart and Batchler (2012).

### 6.3.4 Steel Arches and Sets

Tendon support systems (rock bolts and cable bolts) have largely replaced steel arches and rectangular steel sets (steel cross supports

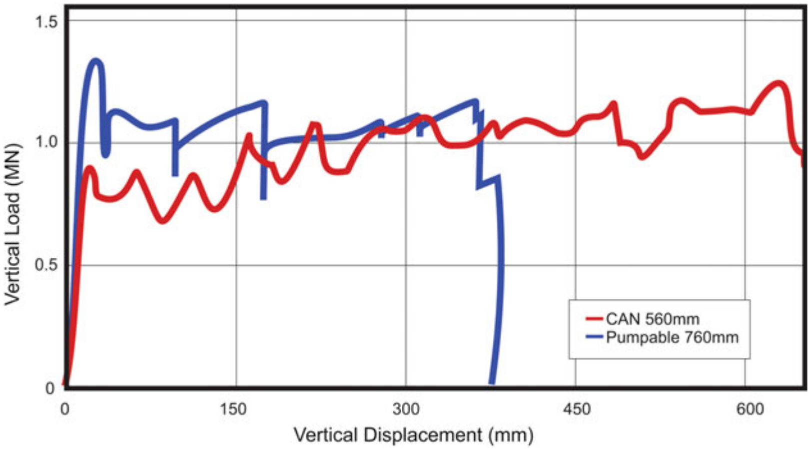

**Fig. 6.13** Load-displacement characteristics for a CAN® and a pumpable cementitious crib (Adapted from Dolinar 2010)

mounted on steel legs) as a primary support system in modern mines. However, steel arches and sets still find application in situations where it is not possible to achieve anchorage or significant load transfer using tendons and where dead weight load must be transferred to the excavation sides and floor. These situations are most often associated with mining through geological disturbances such as faults, dykes and diatremes and in recovering roof falls. As rigid arches and sets can tolerate only a small amount of displacement before being permanently deformed, yielding arches should be considered in situations of ongoing convergence. If the rock is highly fractured or friable, the gaps between each arch usually need to be lagged with timber planks, metal sheets or welded mesh (Fig. 6.14). More detailed information on the capacity, selection and installation of steel arches is to be found in Hoek and Brown (1980) and Gale et al. (1993).

### 6.3.5 Pillars

Pillars are the most effective measure for controlling convergence because at no time is confinement removed from the ground they are required to support; they are stiff; they have a high load carrying area; and a very high support capacity. A 2.7 m high, 4 m square coal pillar, for example, has the potential to sustain a load of around 120 MN (12,000 t) at less than 0.5 % strain (or 10 mm compression). These benefits are apparent when the load-displacement characteristics of the various standing support systems are compared as shown in Table 6.2.

## 6.4 Tendon Support and Reinforcement

### 6.4.1 Scope

The term **tendon** is sometimes used to denote a cable that has been pretensioned, as distinct from an untensioned cable, or dowel. In this text, the term refers to all forms of bar, tube, and wire strand (cable) ground support systems that are anchored to the rock mass in drill holes, irrespective of whether they are pretensioned. The most common types of tendon are:

- wooden dowels;
- steel, plastic and fibreglass rock bolts, either of solid or tubular construction;
- civil construction steel reinforcement bars (rebar);
- cable bolts, which comprise one or more strands of wire, and include wire ropes, multiple strand cables and multiple wire bolts;
- split steel tubes; and
- expandable steel tubes.

Tendons can be classified into one of four category headings, shown in order of increasing reinforcement capacity in Table 6.3.

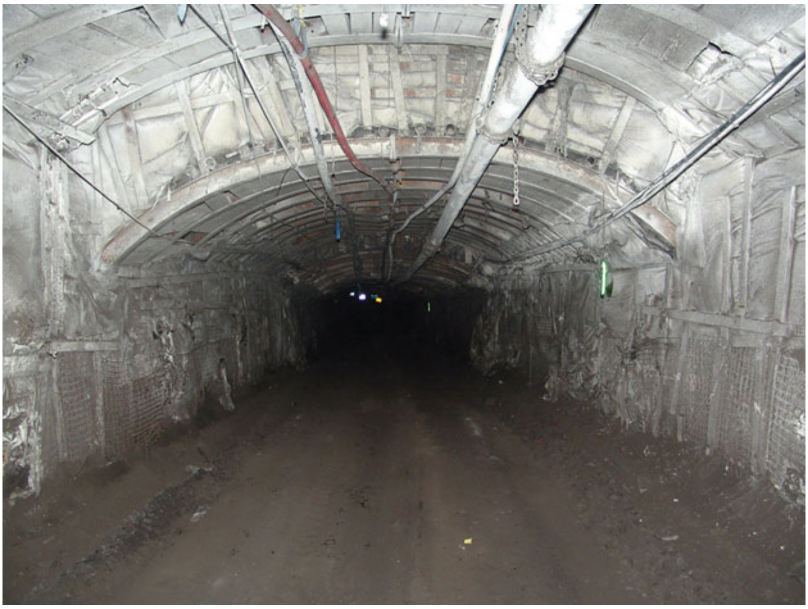

**Fig. 6.14** Steel arches, lagging and rib replacement used to support a severely disturbed geologically zone

**Table 6.2** Comparison between peak load capacity and associated strain for various support/reinforcement systems in a 2.7 m high roadway

| Support/ Reinforcement system | Typical strain required to generate peak load | Peak load capacity (MN) | Approximate number of uniformly loaded supports required to generate the same support resistance as a 4 m square coal pillar |
|---|---|---|---|
| 4 m square coal pillar | 0.3 % | 120 | 1 |
| Hardwood Prop | 1 % | 0.5 | 240 |
| 4 Point Chock | 2 % (yield) | 1 (yield) | 120 |
| | 13 % (ultimate) | 2 (ultimate) | 60 |
| Link-n-Lock® | 2 % (yield) | 2 (yield) | 60 |
| | 4 % (ultimate) | 3 (ultimate) | 40 |
| Cementitious Column | 1.5 % | 2–3 | 40–60 |

**Table 6.3** Classification of tendons (extended from Windsor and Thompson 1997)

| | | Typical length | |
|---|---|---|---|
| Tendon classification | Nature of instability | General application | Underground coal mining |
| Dowel | Surface | 0–3 m | 1.0–1.5 m |
| Rock bolt | Surface | 0–3 m | 1.2–2.7 m |
| Cable bolt | Near surface | 3–15 m | 4–12 m |
| Ground anchor | Deep seated | 10–30 m | Rarely used |

There is mention in the literature of rock bolting having been practiced in the USA before the turn of the twentieth century (Gardner 1971). According to Bieniawski (1987), a description of the planned systematic use of rock bolts first appeared in the literature in 1943. This was in relation to their use at a lead mine in the USA. By 1949, rock bolts were used in over 200 mines in

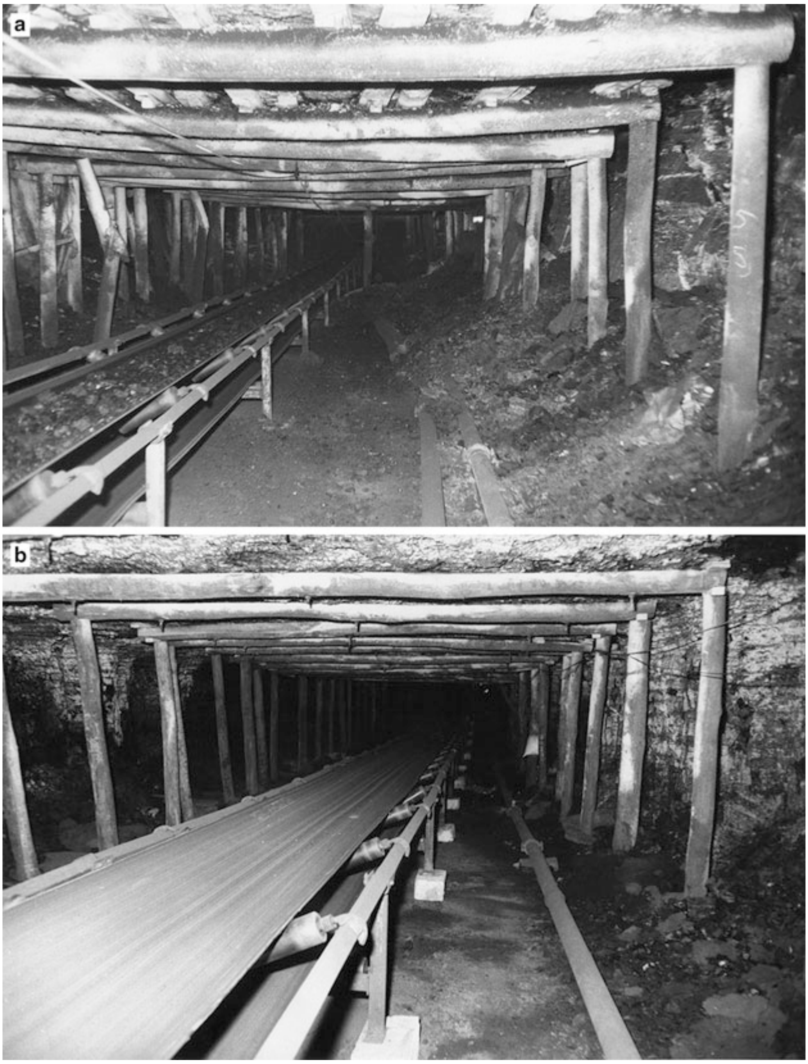

**Fig. 6.15** Change in roof and rib conditions in the Greta Seam, Australia, brought about by the introduction of rock bolting. (**a**) Roof and rib conditions preceding the introduction of roof bolting, (**b**) Roof and rib conditions following the introduction of roof bolting – alternative rows of timber supports reflect mine workers initial concerns about the effectiveness of roof bolts and W straps

the USA, including coal mines. As a result of the experiences of Consolidated Coal in the USA, rock bolts were trialled for the first time in Australian coal mines in 1949 in the Greta Seam at Elrington No. 2 Colliery in Australia (Gardner 1971). These trials involved complementing the standard timber cross supports (baulks) and legs with 2.3 m long, split wedge point anchored bolts, installed three to a row every 1.8 m, through 1.2 m of mudstone into coal. Benefits were immediate (McKensey 1952) and rock bolts were progressively introduced into the NSW coal industry from that time, although it was to take another 25 years for miners to accept the total removal of timber props and cross supports as primary support measures. The benefits offered by rock bolts in mining the Greta Seam are apparent in Fig. 6.15 and significantly, included better rib control.

Tendons reinforce the rock mass primarily by providing:

- a clamping force across parting planes to resist bed separation and slow down displacement; and
- a physical obstruction to shear displacement along parting planes.

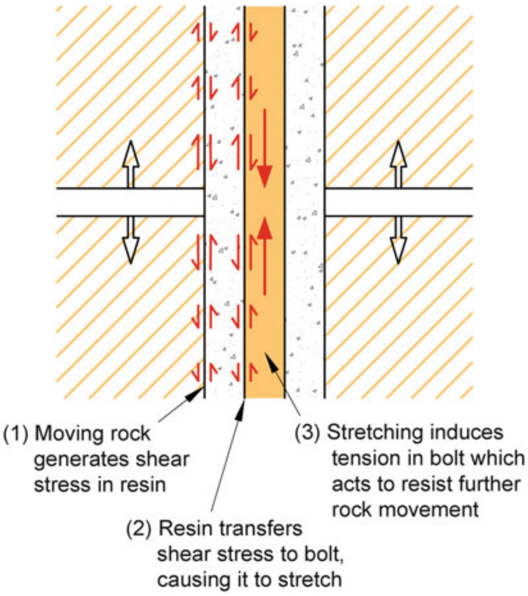
(1) Moving rock generates shear stress in resin
(3) Stretching induces tension in bolt which acts to resist further rock movement
(2) Resin transfers shear stress to bolt, causing it to stretch

**Fig. 6.16** Mechanism for load transfer between the rock mass and a tendon

The clamping force is provided through an iterative process referred to as **load transfer**. When a tendon is anchored on either side of a parting plane, separation on this plane induces tension in the tendon which then acts as a clamping force to resist further rock displacement (Fig. 6.16). The effectiveness of this system depends on the stiffness of the tendon, the ultimate load capacity of the tendon, and the type and capacity of components used to transfer the load between the rock mass and the tendon.

The three main methods for anchoring a tendon in a drill hole are illustrated in Fig. 6.17, these being:

- a mechanical expansion anchor;
- embedment in a cementitious or resin based grout; and
- friction around the perimeter of the tendon over its full length.

Tendons that are anchored only at the back of a drill hole are referred to as 'point anchored' or 'end anchored' and require a face plate with a retaining mechanism in order to generate a reaction force to rock dilation. In underground coal mining, it is usual practice to fit all tendons with a face plate that is restrained by a forged stud, a threaded nut or a barrel and wedge arrangement. This is for the purpose of supporting the surface skin of the excavation, applying pretension to the tendon, and/or maintaining or increasing tension in the tendon by resisting it being pulled into the drill hole as the strata dilates. Exceptions sometimes occur in the case of long cable bolts.

Tendons may comprise discrete reinforcement units or elements of an integrated system that includes surface support elements such as:

- steel cross supports, usually light gauge 'W' straps but, on occasions, rolled steel joists (RSJs) and customised profiles (top hats);
- steel and synthetic mesh screen;
- wire slings, constructed by tying and tensioning the tails of cable bolts;
- trussing, comprising tensioned steel bars linking pairs of tendons;
- rope lacing; and
- sprayed cementitious and synthetic based membranes or liners.

The performance of a tendon support system is influenced by many variables, including:

- lithological sequence of the rock mass;
- structural fabric of individual rock layers;
- rock mass strength properties;
- in situ stress regime;
- tendon type, length and density;
- tendon material properties;
- tendon anchorage mechanism;
- load transfer characteristics and capacities of the tendon and its anchorage system;
- stress directions relative to mining direction;
- elapsed time to installation;
- rock mass failure mode.

Underground coal mining is commonly associated with bedded and/or laminated immediate roof strata. This environment is conducive to normal and shear displacements developing at multiple horizons within the immediate roof, with the magnitude and direction of these movements varying along the width and length

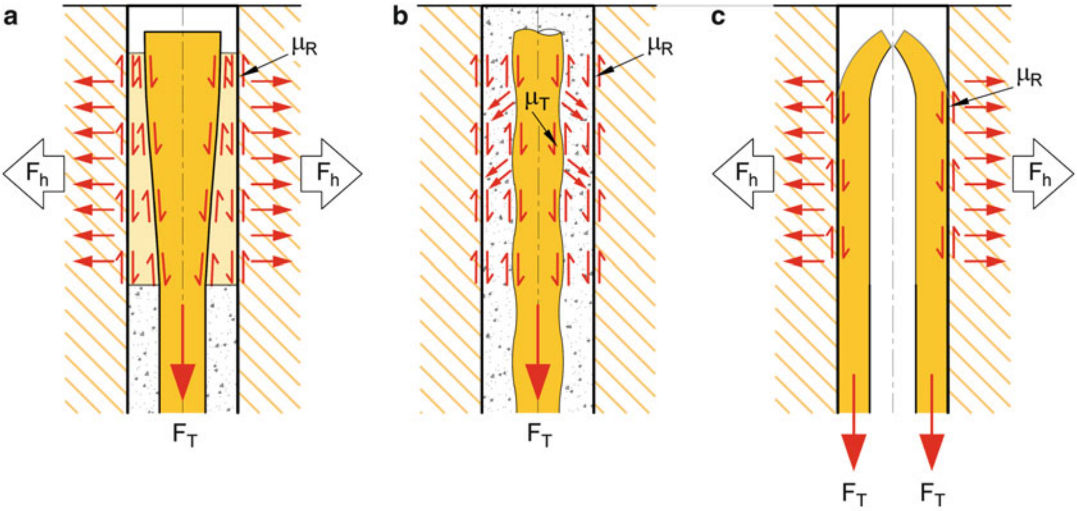

**Fig. 6.17** Techniques for anchoring a tendon inside a drill hole. (**a**) Mechanical, (**b**) Grouted, (**c**) Friction

of the excavation. In conjunction with rock mass behaviour, this represents a particularly complex load-response environment. Fully encapsulated tendons are the most extensively adopted support for these conditions.

In theory, the design of a tendon system should be based on knowledge and understanding of rock mass behaviour in the specific conditions; mode of rock deformation in the vicinity of excavation walls; the behaviour of the tendon system; and the interaction between these factors in the given conditions. In practice, the range of variables and the complexity of interactions between them make this unachievable other than in very simple loading environments. This text is focused on presenting the basic principles and factors that research and field experience have identified as underpinning the performance of tendons, in order to assist practitioners in developing a holistic understanding of ground behaviour and reinforcement.

## 6.4.2 Functions of Tendons

The primary functions of tendons are to:

- maintain and enhance the strength properties of jointed rock mass;
- prevent strata separation; and
- control the consequences of post-failure deformation.

These are achieved utilising four basic mechanisms, which may be interactive or overlapping to some degree, namely:

- suspension;
- beam building in laminated strata;
- keying and pressure arch formation in fractured and blocky rock masses; and
- confinement of failing and broken strata.

The last three mechanisms are all concerned with mobilising and enhancing the self-supporting capabilities of the rock mass in the fracture zones around an excavation.

### 6.4.2.1 Suspension

Suspension involves pinning loose slabs of rock and weak layers of rock to more competent strata in the roof and sidewalls (ribs or ribsides) of an excavation. Hence, the tendons function as a form of support rather than reinforcement. Examples of where tendons find application for this purpose are illustrated in Fig. 6.18. In cases where a loose slab constitutes a keystone, the mechanism can overlap with that of pressure arch building. Because deadweight primarily generates the forces in the support system, the

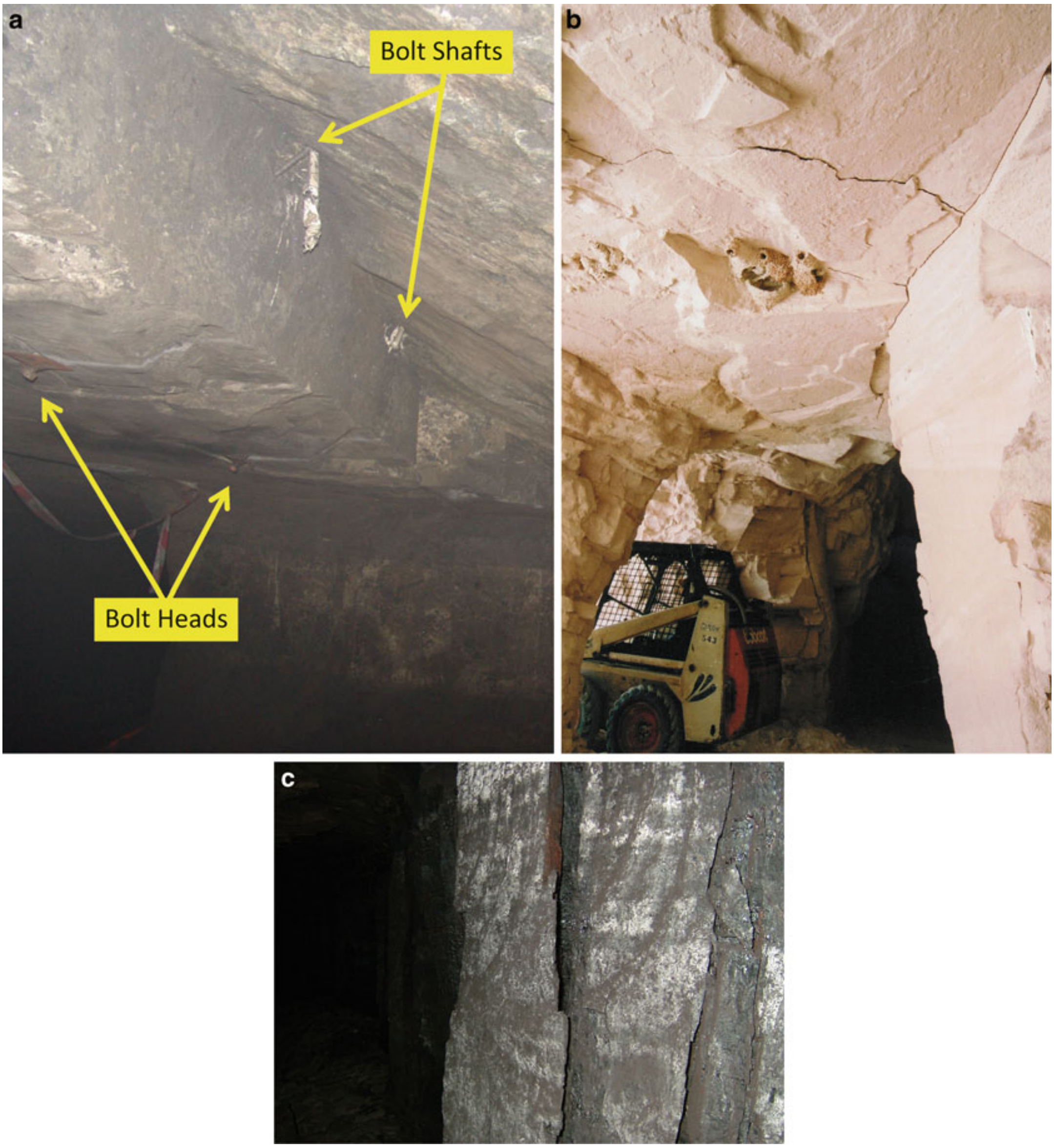

**Fig. 6.18** Examples of suspension applications for tendons. (**a**) Roof fall in which tendons only intercepted the corner of the block that required suspension, (**b**) Immediate roof in need of being suspended off tendons, (**c**) Coal sidewall in need of pinning to pillar core

loading system is load controlled. This is sometimes also referred to as ‘weight controlled’.

The tendon, the anchor and the collar components each need to be designed to sustain the vertical component of the weight of the suspended rock mass and the lever arms (or bending moments) associated with any cantilevered slabs of rock. Bending moments are often overlooked and are best managed by placing tendons in a pattern that supports lips and brows. Design also has to consider the capacity of the suspended rock layers to bridge between tendons. This can be achieved with judicious tendon spacing and supplementary surface support systems. Design safety factors typically range from 1.5 to 2.0, depending on exposure of personnel, the importance of the excavation, geological variability, and confidence in the capacity of each support system element.

### 6.4.2.2 Beam Building

It is often the case when the immediate roof is bedded or laminated that there are no competent beds within several metres of the roof from which to suspend thin rock layers. Roof support is reliant, therefore, on constructing a laminated beam in an environment where bedding planes are characterised by low to zero tensile strength normal to the bedding planes and low shear strength along the bedding planes.

When a roof beam deflects, the upper surface is shortened and the lower surface is lengthened. Hence, in order for a laminated roof to sag, the beds have to slide past one another, as illustrated simply in Fig. 6.19 for the case of a suite of simply supported beams. The principle of beam building is to prevent this sliding movement by increasing the effective thickness of the beam.

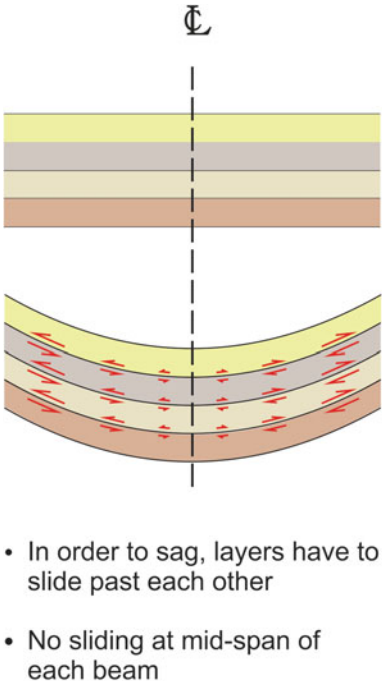
℄
• In order to sag, layers have to slide past each other
• No sliding at mid-span of each beam

**Fig. 6.19** Diagrammatic representation of how beds have to slide past each other in order for the roof to sag, and the relative shear displacement that results

In the ideal world of classical beam theory, the vertical centre line of the sagging roof strata is a plane of symmetry and so there is no shear displacement between bedding surfaces along this plane. Shear displacement increases with distance from the centre of the individual beams towards the abutments. It follows from Fig. 6.19 that, firstly, the construction of a laminated beam is concerned with increasing the shear strength of the bedding planes and, secondly, that the greatest benefits are to be had by focussing these efforts towards the beam abutments where shear stresses are a maximum.

The reader is referred to Sect. 2.8 for a description of the fundamental principles that govern beam behaviour and shear displacement. Based on these principles, it follows that the shear strength of bedding planes can be increased by:

- Increasing the clamping forces across the planes in order to both keep the surfaces in close contact so that the contact strength of surface asperities and undulations can be utilised to resist shearing and to increase the friction between the faces of surfaces that are in contact. In practice, this is achieved by, firstly, enhancing mechanical interlock by taking advantage of hydraulic temporary roof support systems to jack up the roof prior to setting the tendon anchor in a process colloquially referred to as **thrust bolting**; secondly, by applying a pretension to a tendon; and, thirdly, by additional clamping forces generated in a tendon in response to shear movement.
- Increasing the cohesive strength of bedding planes. This is achieved primarily through the dowelling action of tendons, which act as obstructions to shear displacement. The horizontal component of force generated in a tendon in response to shear movement also makes a contribution to increasing effective cohesion.

The concept of beam building from a mechanistic perspective is illustrated in Fig. 6.20 using a simple case of four beds of equal thickness, t,

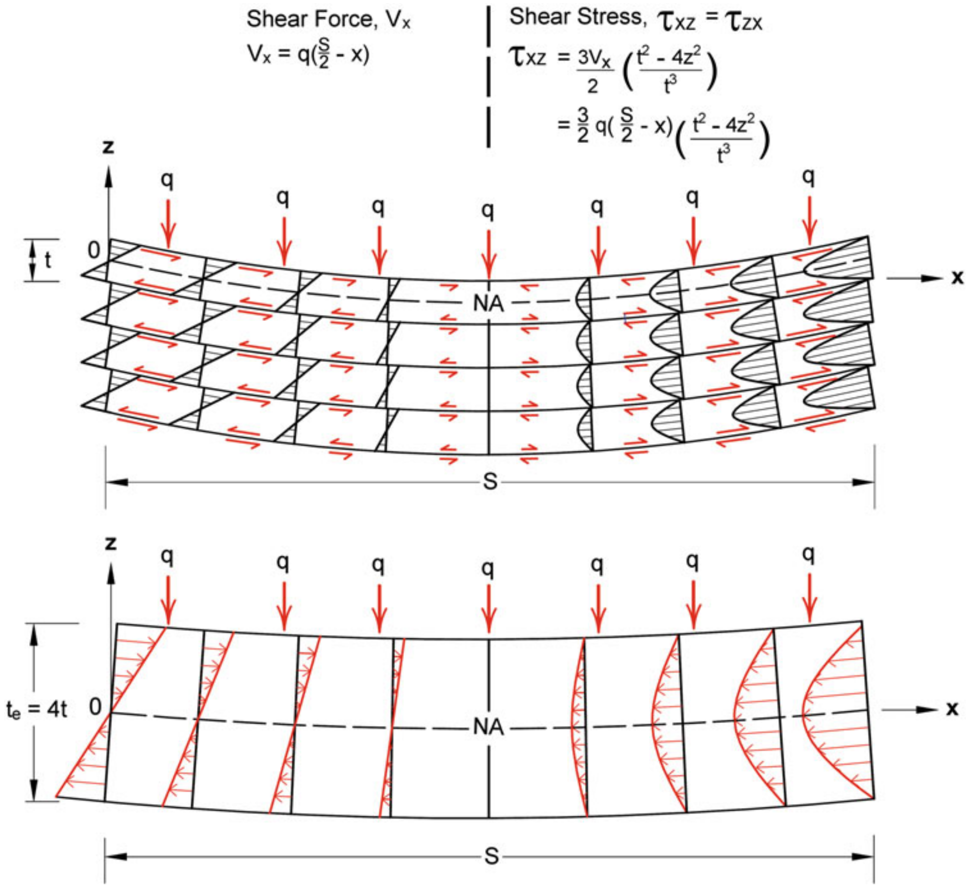
Shear Force, $V_x$
$V_x = q(\frac{S}{2} - x)$
Shear Stress, $\tau_{xz} = \tau_{zx}$
$\tau_{xz} = \frac{3V_x}{2}\left(\frac{t^2 - 4z^2}{t^3}\right)$
$= \frac{3}{2} q(\frac{S}{2} - x)\left(\frac{t^2 - 4z^2}{t^3}\right)$
z
q
t
0
NA
x
S
$t_e = 4t$

**Fig. 6.20** The concept of beam building, showing the effect on shear force and shear stress distribution

that possess the same mechanical properties. Effectively, by preventing sliding on the plane between the lower two beds, this plane is turned into the neutral axis of a beam of thickness 2t, thereby transforming it from the plane of minimum shear stress in each single beam, to the plane of maximum shear stress in the composite beam. The process continues up through the beds to result in a composite beam of effective thickness, 4t, resulting in a 16 fold decrease in deflection.

The effectiveness of placing tendons close to the beam abutments has been demonstrated in physical model tests by Spann and Napier (1983), summarised in Fig. 6.21, and by Stimpson (1987). This effectiveness is particularly apparent in Fig. 6.21 when beam deflection for bolting pattern No. 4 is compared with that for bolting patterns No. 8, 9 and 17. Increasing support from 4 bolts in pattern 9, to 11 bolts in pattern 17, by locating the additional bolts within the existing bolting pattern made little difference to beam deflection.

When rock is subjected to bending, its outer surface is placed into tension. Flexural strength is a measure of the ultimate tensile strength of the outer fabric of a material subjected to bending and is often higher than the overall uniaxial tensile strength of the material. In the case of rock, flexural strength can vary within a wide range but is reported to be typically about 1/20 of the uniaxial compressive strength (Wagner 1985a) and to have the average values given in Table 6.4 (Wagner 1994). Wagner (1985a) applied lower bound values of flexural strength to show that, theoretically, in the absence of local structural

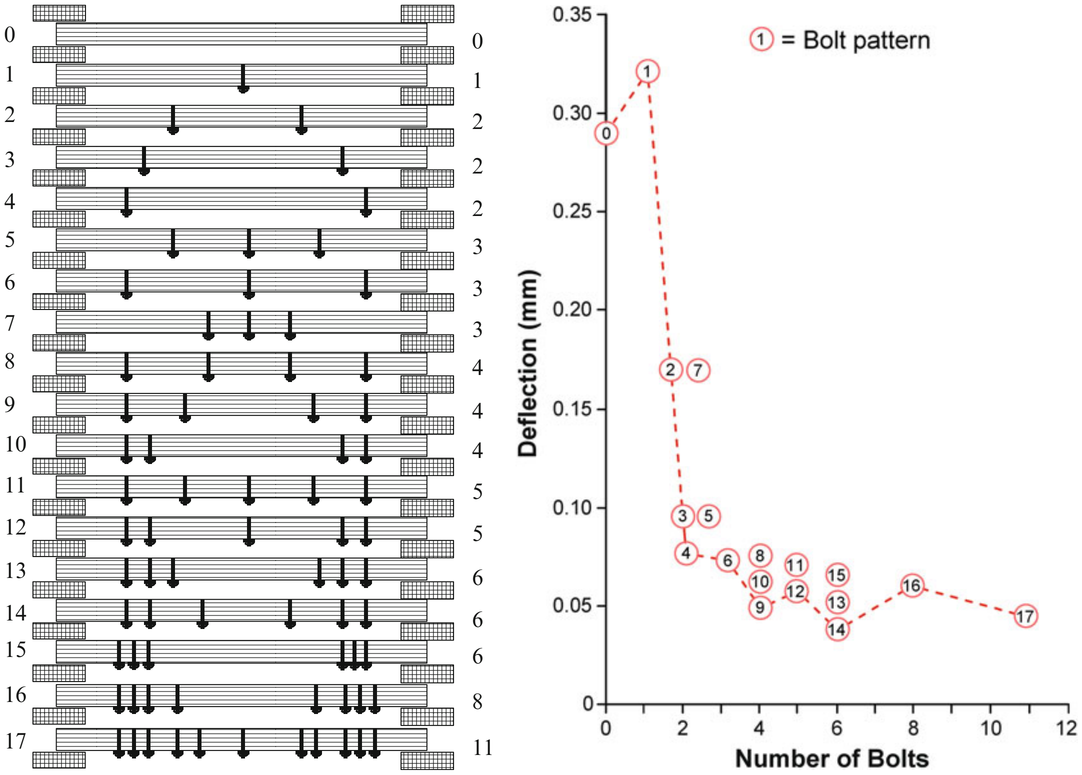

**Fig. 6.21** Model test results demonstrating the effect of different bolting patterns on beam deflection (After Spann and Napier 1983)

**Table 6.4** Strength values for typical coal measure strata (After Wagner 1994)

| Strata | Modulus of elasticity (GPa) | Typical uniaxial compressive strength (MPa) | Flexural strength (MPa) |
|---|---|---|---|
| Sandstone | | | |
| Fine grained | 9 | 70–120 | 5–9 |
| Medium grained | 6 | 50–70 | 3–6 |
| Course grained | 4 | 30–50 | 2–4 |
| Shale | 3 | 60–80 | 2–4 |
| Coal | 2 | 15–40 | 1.5–2.5 |

weaknesses, a coal roof beam of about 0.45 m in thickness could be expected to be self supporting across spans up to 7 m, as shown in Fig. 6.22. The sensitivity of roof stability to excavation span is illustrated by a halving in the required thickness of the coal beam when excavation width is reduced from 7 to 5 m.

The type, number, length, angle, location, spacing and pretensioning of tendons are all factors which need to be considered when designing tendon reinforcing systems. An understanding of their contribution is facilitated by firstly considering the simplest situation of vertical tendons on the brink of being subjected to shear displacement. This provides the foundations for then considering the response of tendons to significant shear displacement. In analysis of this type, the amount by which shear stress exceeds shear resistance is sometimes referred to as **excess shear stress** or **excess bedding-shear stress**.

**Interface cohesion**, c, is usually very low in sedimentary environments and often taken to be zero, as assumed in the analysis that follows. The coefficient of friction, μ, varies with the nature of

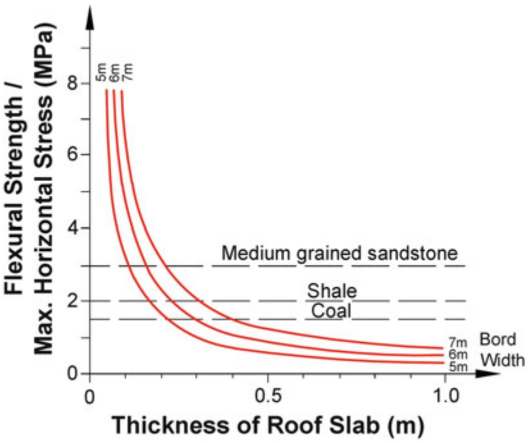

**Fig. 6.22** Bending stresses in roof beams of different thickness for various excavation widths (After Wagner 1985a)

the strata, typically ranging from 0.3 for slickensided material to 1 for sandstone. The normal stress, $\sigma_n$, across an interface depends upon the ratio of the load per unit length to the flexural rigidity, EI, for each layer. If this ratio is less for the upper beam than for the lower beam, then the two will act independently as single beams. If the reverse is the case, then the upper beam will load the lower beam. Appendix 3 provides the load and displacement equations pertaining to this latter case. For present purposes, the beams are assumed to act in a dependent manner.

Applying tension to a tendon increases the clamping forces across an interface, thus increasing the frictional resistance to sliding. The increase in shear resistance per square metre of roof, $\tau_f$, is given by:

$$\tau_f = nF_{pt} Tan\ \phi = nF_{pt}\mu \tag{6.1}$$

where

$n$ = *number of tendons per square metre*
$F_{pt}$ = *tensile force (pretension) applied to tendon*
$\mu$ = *coefficient of friction between interfaces*

This means, for example, that the frictional component of shear resistance generated by tendons spaced 1 m apart on a 2 m row spacing and pretensioned to 50 kN ranges from 7.5 to 25 kPa for a 0.3–1 range in coefficient of friction.

In order for shear movement to occur, the shear resistance generated by the doweling action of the tendons also has to be overcome. This resistance is proportional to the number of tendons installed per square metre of roof and to the shear strength of the tendons, the embedment medium and the rock mass.

To assist in developing concepts, assume for the moment that the embedment medium and surrounding rock mass are rigid (infinitely stiff), so that the cohesive benefit of a tendon is determined solely by its shear strength. Since the shear strength of a steel tendon is typically 50 % of its ultimate tensile strength, the cohesive component, $\tau_c$, of shear resistance is given by:

$$\tau_c = 0.5nS_t \tag{6.2}$$

where

$n$ = *number of tendons per square metre*
$S_t$ = *ultimate tensile strength of the tendon*

Based on this equation, the cohesive component of shear resistance generated by 300 kN (30 t) tendons spaced 1 m apart on a 2 m row spacing is of the order of 75 kPa. Therefore, it follows if significant shear displacement is yet to occur in this example case, the cohesive component of shear resistance would make a much greater contribution than the frictional component to the total shear resistance, $\tau_T$, provided by a tendon.

The total shear resistance per square metre of roof in this example is given by Eq. 6.3.

$$\tau_T = n\left(\mu F_{pt} + 0.5S_t\right) \tag{6.3}$$

Figure 6.23 summarises the effects of roof bolt density and coefficient of friction between interfaces on the total shear resistance generated by a tendon based on Eq. 6.3 for this simple case in which the rock mass is assumed to be rigid. The left hand side shows that one 190 kN (19 t) capacity tendon per square metre tensioned to 50 kN has the capacity to increase shear

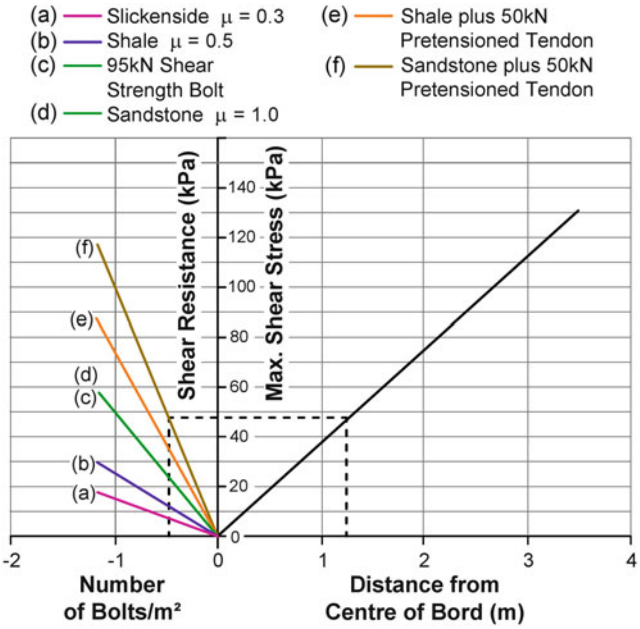

**Fig. 6.23** Effect of coefficient of friction, tendon location and tendon density on shear resistance (Adapted from Wagner 1985b)

resistance by a factor of 2–3. The right hand side shows the increase in shear stress at the neutral axis of a roof beam with distance from the midspan of an excavation. Correlation between the two demonstrates the need to increase bolting density with distance from the centre of an excavation. Important conclusions that find general application and that can be drawn from Fig. 6.23 are (Wagner 1994):

- Tendon density required to prevent shear movement along bedding planes is strongly dependent on the type of roof strata, with strata with a lower coefficient of friction requiring a higher density of reinforcement.
- Tendon density should be increased towards the excavation abutments, where shear stresses are highest. The concept of each tendon doing an equal amount of work by being installed at the centroid of shear force segments of equal area provides a basis for this design (Fig. 6.24).
- Fully encapsulated tendons are superior in laminated strata since their resistance to sliding becomes effective immediately the encapsulation medium sets.

The situation becomes more complex when the rock mass and embedment medium are no longer assumed to be rigid and significant shear displacement develops. Although the shear strength of a tendon is typically 50 % of its ultimate tensile strength, a range of studies has shown that the total shear resistance generated by a fully encapsulated tendon can be considerably higher than this, being of the order of 90 % of ultimate tensile strength in some instances (Azuar 1977; Canbulat 2008; Craig and Aziz 2010). This behaviour is generally attributed to the additional clamping forces generated by the tendon as a result of it becoming elongated during shearing as shown in Fig. 6.25.

The magnitude of these additional clamping forces is a function of the stiffness of the tendon and the behaviour of the localised zone of deformation that develops around it (Fig. 6.26). The extent of the deformation zone is a function of the deformation properties of both the rock and the tendon, the diameter of the tendon, the type of tendon (hollow or solid) and the amount of shear displacement. This is shown in Fig. 6.27 in respect of the indentation characteristics of the

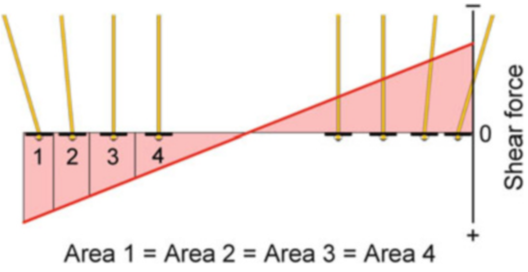

**Fig. 6.24** Design concept whereby each tendon is subjected to an equal shear force

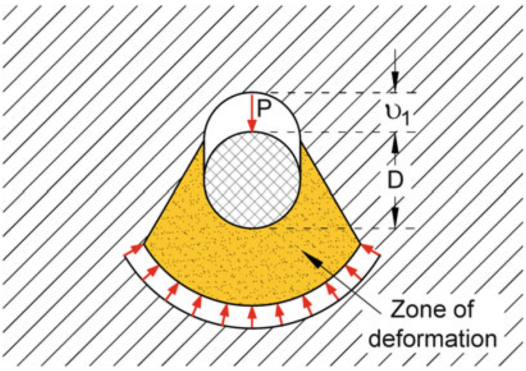

**Fig. 6.26** Rock deformation around a tendon subjected to shear displacement (Adapted from Schubert 1984)

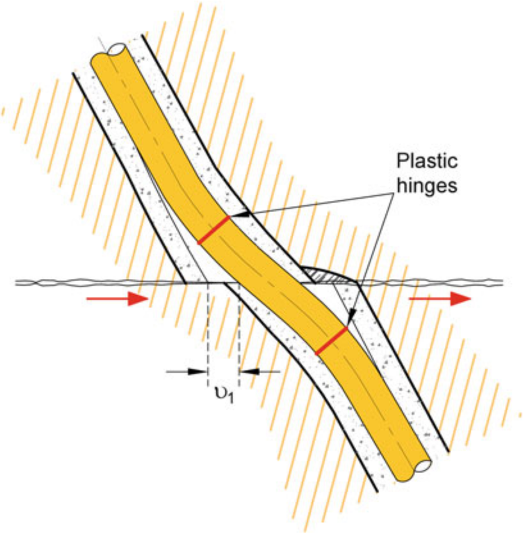

**Fig. 6.25** Generic model of the response of a fully encapsulated tendon to shear displacement

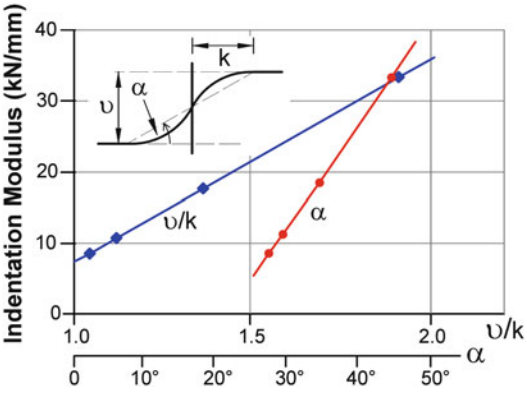

**Fig. 6.27** Drill hole deformation due to shear displacement along a fracture plane (After Schubert 1984)

rock. In the figure, the parameter ‘$\upsilon$’ is shear displacement and the parameter ‘k’ is the distance from the shear plane over which drill hole deformation has occurred. The softer the rock, the greater the zone of deformation, with curvature of the tendon being less and extending for a greater distance into the rock mass.

The influence of tendon diameter on shear resistance is reflected in Fig. 6.28. Shear resistance increases with both increasing tendon diameter and increasing shear deformation, with the latter generating an increase in clamping force as the tendon is stretched. As a result of this mechanism, the resistance to shear generated by a tendon can be of the same order of magnitude as its resistance to axial deformation, despite the shear strength of steel being only about one-half that of its tensile strength. It follows that the coefficient of friction of a joint plane is a critical parameter in situations of significant shear displacement.

Similar conclusions have been drawn by others. Pells (2008) employed a model closely resembling that shown in Fig. 6.25 to evaluate the relative contribution of five factors that result in an increase in shear resistance of a joint when a fully encapsulated and pretensioned tendon is installed at an angle across the joint plane. These factors are listed in Table 6.5, with their total contributions defined (in terms of stress contribution per tendon) by Eq. 6.4. The relative contribution of each factor is plotted in Fig. 6.29 for

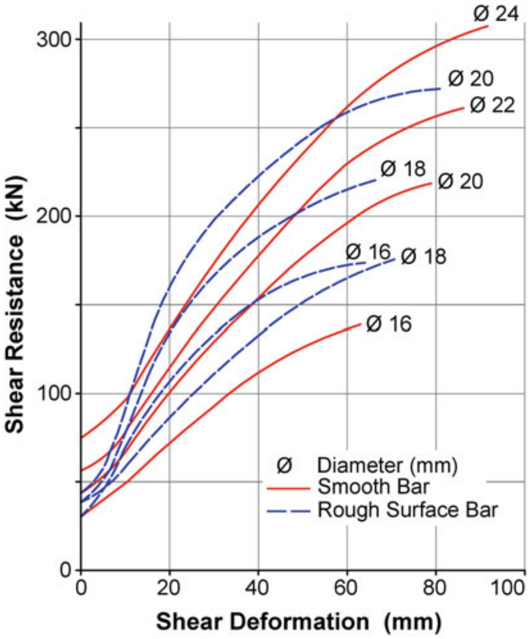

**Fig. 6.28** Influence of bolt diameter and surface finish on shear stiffness of solid bar rock bolts (Adapted from Schubert 1984)

the case of a 25 mm diameter bar installed across a typical joint in Hawkesbury sandstone.

$$S_j = \left(c_j + \Delta c\right) + \left(\sigma_{no} + \Delta\sigma_n\right) tan\, \phi_j \qquad (6.4)$$

where

$S_j = equivalent\ shear\ strength\ of\ the\ joint$

$\Delta c = \dfrac{R_1 + R_5}{A}$

$\Delta\sigma_n = \dfrac{R_2 + R_3 + R_4}{A}$

$A = area\ affected\ by\ each\ bolt$

$c_j = \ effective\ cohesion\ of\ joint$

$\phi_j = effective\ friction\ angle\ of\ joint$

$\sigma_{no} = effective\ normal\ stress\ on\ joint$

$\Delta c = equivalent\ increase\ in\ effective\ cohesion$

$\Delta\sigma_n = equivalent\ increase\ in\ effective\ stress$

The modelling undertaken by Pells (2008) indicated that friction associated with joint roughness made the most significant contribution to shear strength in the given circumstances. Seedsman (2012a) made a similar finding based on analytical modelling, concluding that frictional resistance can be 3–4 times greater than dowel resistance. These findings highlight the need in bedded and laminated roof environments to install tendons as close as possible to the face to prevent partings from opening. They largely account for the success of thrust bolting. The modelling outcomes of Pells (2008) also give insight into the benefits of pretensioning, with force $R_2$ and most of force $R_5$ shown in Fig. 6.29 being attributable to pretensioning of the tendon.

**Table 6.5** Actions associated with a fully encapsulated tendon that contribute to an increase in shear strength

| Source of increase in shear resistance | Force | Component |
|---|---|---|
| Increase in lateral resistance developed by tendon via dowel action | $R_1$ | Cohesion |
| Increase in normal stress due to pretensioning or prestressing of the tendon | $R_2$ | Friction |
| Increase in normal stress due to axial force developed in the tendon from dilatancy of the joint | $R_3$ | Friction |
| Increase in normal stress due to axial force developed in the tendon from lateral extension | $R_4$ | Friction |
| Increase in shear resistance due to the axial force in the tendon resolved in the direction of the joint | $R_5$ | Cohesion |

A review by Hartman and Hebblewhite (2003) of research into shear loading under laboratory conditions concluded that there appears to be general agreement that shear resistance is a function of the inclination of a tendon to a sliding plane. The findings of Windsor and Thompson (1993), plotted in Fig. 6.30, are particularly significant because they not only show that angled bolts generate higher shear resistance when they are put into tension by sliding, but also that bolts angled in the opposite direction and put into compression by sliding generate a much lower resistance than bolts installed vertically. Pells (2008) also reported that bolts inclined across bedding planes against the direction of shear can be ineffective. This has important implications for continuous miner-mounted drill rigs designed to install the outer tendons angled towards the ribline and the inner tendons angled towards the centre of the roadway.

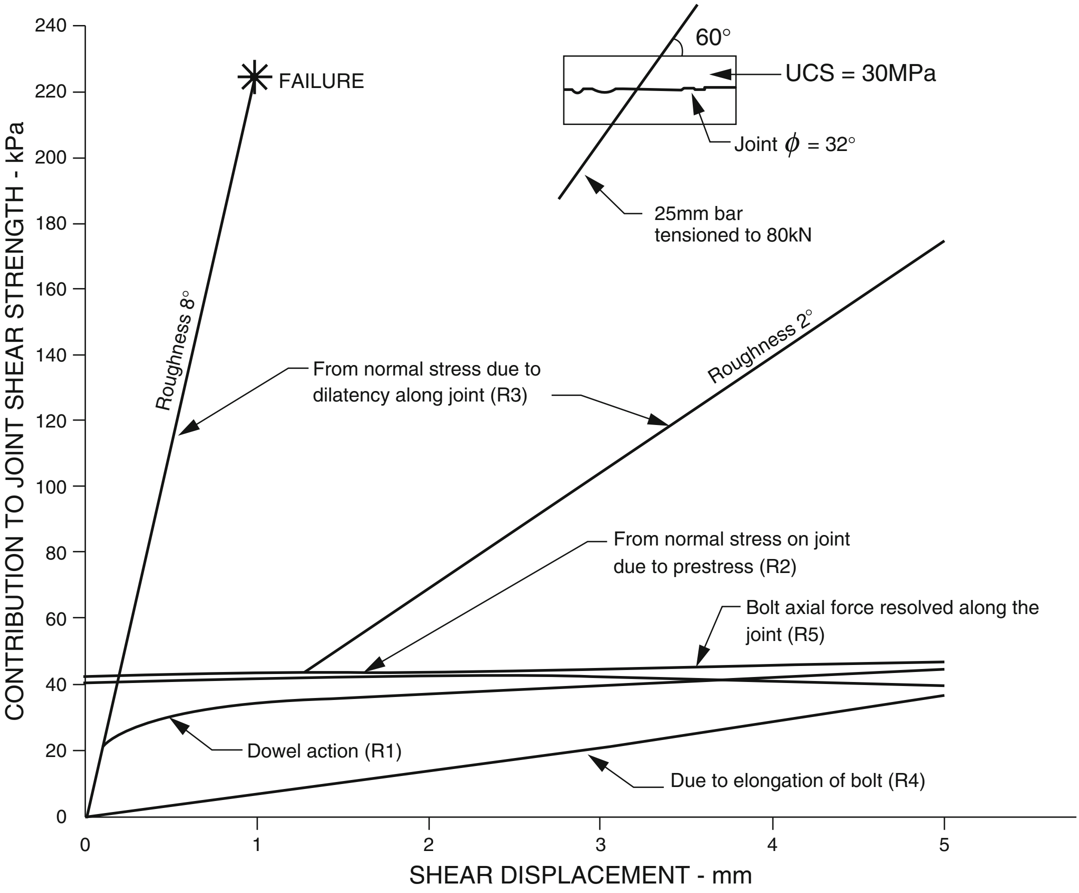

**Fig. 6.29** Calculated contribution of different tendon actions to joint shear strength in Hawkesbury sandstone (After Pells 2008)

The behaviour of angled tendons is conceptualised in Fig. 6.31. A field example of increased effectiveness when angled towards the ribline is illustrated in Fig. 6.32. Tendons installed close to the ribline and angled over it are more effective in controlling bending stresses and roof sag in a laminated environment because they penetrate the zone of highest shear stress close to the roof and because they generate higher axial loads in response to displacement on sliding planes. Long angled tendons anchored over the excavation abutments offer a significant additional benefit in enabling the deadweight of the immediate roof strata to be suspended off the abutments if roof control is lost, as illustrated in Fig. 6.33. These tendons usually comprise cables integrated with some form of cross support.

The effectiveness of cable tendons is reflected in their expanded use as primary and secondary support. Initially, cable bolts in underground coal mining were used primarily as secondary support. In adverse roof conditions in Australian mines, it has become common practice to install 10 m long double cables on a 2/1/2 pattern. Sometimes, the 2 outer cables are angled over the rib and trussed to resist bedding plane shear and to cradle and suspend failed roof, as illustrated in Fig. 6.33. The centre cable is installed to assist in controlling roof deflection (discussed in Sects. 7.3.2 and 7.5.3). In some mines, the outer bolts in each row are also angled towards the ribline.

A breakthrough in roof control was made in 1993 at Angus Place Colliery, Australia, when

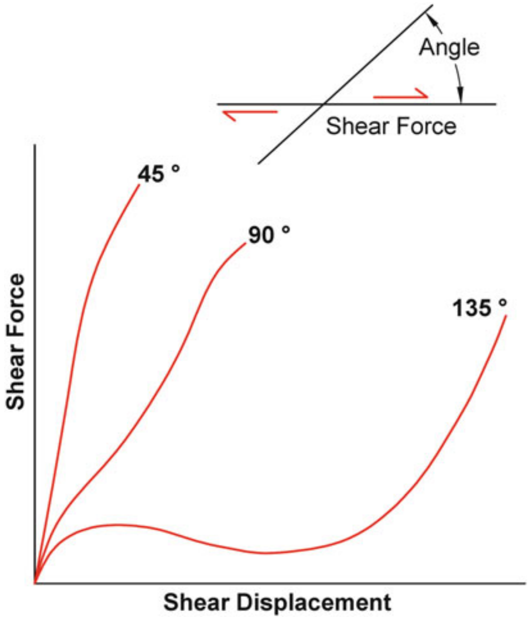

**Fig. 6.30** Effect of tendon inclination on shear resistance generated by the tendon (After Windsor and Thompson 1993, Copyright Elsevier)

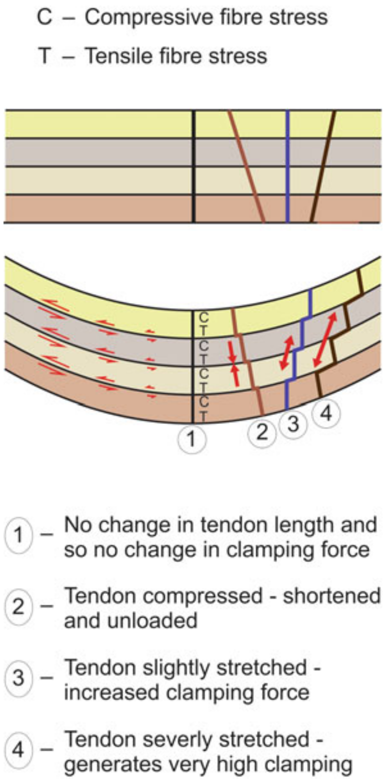
C – Compressive fibre stress
T – Tensile fibre stress
C
T
C
T
C
T
C
T
1
2
3
4
1 – No change in tendon length and so no change in clamping force
2 – Tendon compressed - shortened and unloaded
3 – Tendon slightly stretched - increased clamping force
4 – Tendon severly stretched - generates very high clamping

**Fig. 6.31** Diagrammatic representation of the effect of beds sliding past each on tendon length and deformation relative to tendon distance from the abutments

cables were introduced as primary support at the coal face, enabling the mine to regain control of the roof (Galvin 1996). The cables comprised 4 m long, 500 kN (50 t) capacity Flexibolt brand cables which had a higher shear resistance than rigid X-grade bar rock bolts and a higher ultimate load capacity (Fuller and O'Grady 1993; Fuller et al. 1994). The cables also proved successful at Ellalong Colliery, Australia, this being attributed to their increased length and superior behaviour in shear in comparison to rock bolts (McCowan 1994). Butcher (1994) reported that prior to the introduction of Flexibolts at Angus Place Colliery, roof softening extended to 3–3.5 m in 5 m wide roadways supported with standard 2.4 m long AX bolts, with deformation rates of up to 100 mm/day. Flexibolts reduced the need for primary roof bolts by 50 % and the need for secondary 10 m long, double strand, cement grouted cable bolts by 75 %.

It can be concluded in respect of beam building that:

- The contact strength of rock asperities and undulations can have a major influence on shear resistance and shear deformation.
- Tendons increase the shear resistance of a joint by:
  - increasing effective cohesion as a result of the dowel action of the tendon;
  - increasing normal stress as a result of pretensioning of the tendon;
  - increasing normal stress as a result of an increase in axial force due to dilation of the joint; and
  - increasing normal stress as a result of the additional axial force developed due to lateral extension of the tendon at the joint plane.
- Bed separation and shear movement of the immediate roof beam can be minimised by installing primary support tendons as early and as close to the face as possible. However, in some situations there may be benefit in allowing the immediate roof to relax to some extent before installing primary or secondary support.
- Fully encapsulated, pretensioned tendons are superior to end-anchored and friction

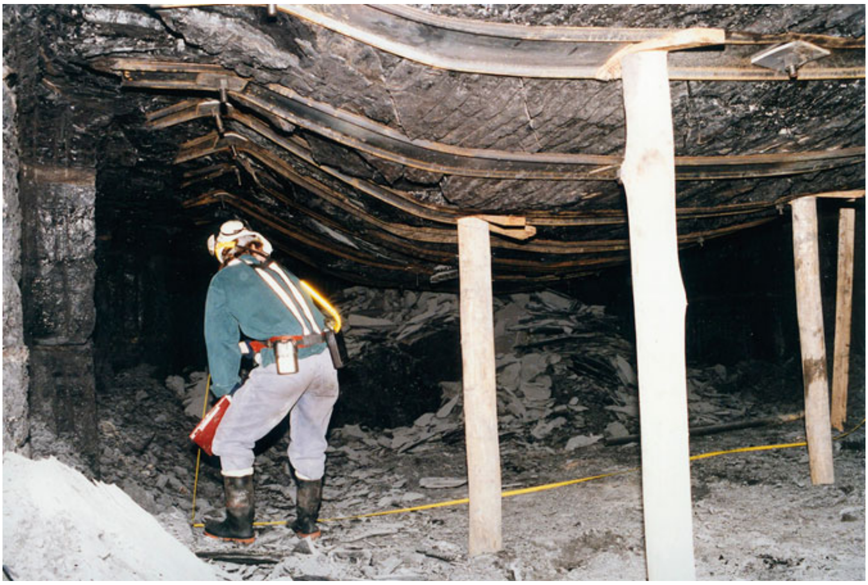

**Fig. 6.32** An illustration of the effectiveness of angled bolts installed towards the abutment in controlling failure of a laminated coal roof

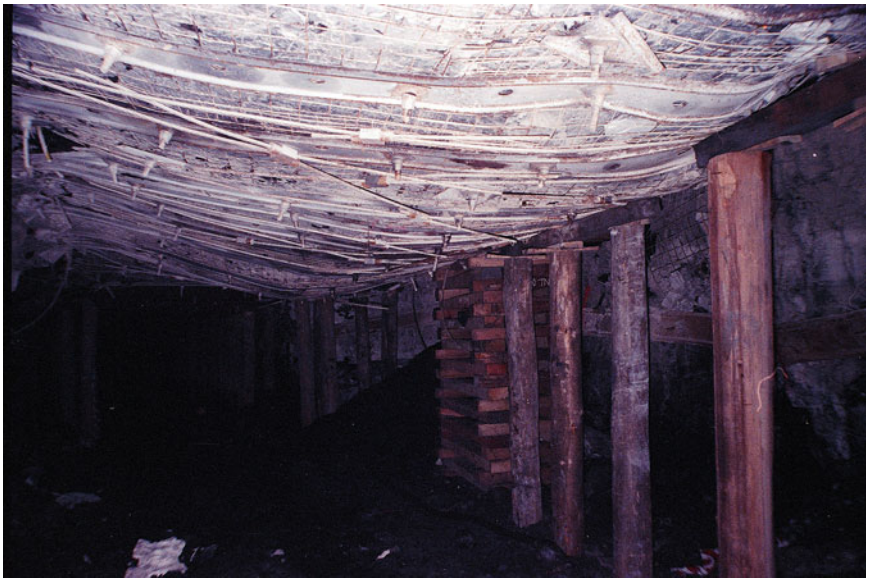

**Fig. 6.33** An example of failed immediate roof suspended off the excavation abutments by means of long tendons angled over the ribline and trussed across the excavation

anchored tendons in resisting shear and enhancing reinforcement.

- Friction bolts can accommodate large shear displacements but the build-up of shear resistance is slower than for other types of tendons.
- Tendon density needs to be higher closer to the abutments than in the centre of the excavation.
- Tendons installed in the middle of a roadway do little to prevent shear but are very important for controlling roof deflection.

- Tendons inclined towards the abutments can enhance reinforcement.
- In situations where strata movement has already taken place, it can be very beneficial to re-establish contact between roof layers. Tensioning of tendons may be inadequate for this purpose and the immediate roof layers may need to be jacked upwards at the time of bolt installation.
- Cable bolts provide a means of installing a longer reinforcing element in a restricted mining height, enabling tendons to be angled over the ribs, to extend to higher heights of softening, and to anchor in more competent strata.

#### 6.4.2.3 Rock Arch Formation

Diagrams such as that shown in Fig. 6.34a often accompany descriptions of how tendons create a rock arch around an excavation by inducing a zone of compression to confine the rock mass. These types of diagrams can create the impression that the immediate roof is subjected to a near uniform zone, or arch, of high compressive stress. In reality, elevated stress bulbs are induced around the reaction points of a tendon, being the face plate and the anchor, but this stress then rapidly dissipates with distance from these points as illustrated by the numerical modelling outcomes of Pells (2008) presented in Fig. 6.34b. The strata between the reaction points are subjected to higher compressive stress but, when averaged over the area, this stress increase may be minimal.

The direct confining effect of the tendons is due principally to the tangential expansion induced by the forces in the tendons. This can have a significant positive influence on rock mass strength. Tendons can also indirectly improve rock mass strength significantly by retaining fractured rock in situ so that this rock resists further deformation and, in so doing, generates confinement to support a rock arch. Pretensioning a tendon results in a given amount of subsequent displacement generating a higher resistance to further displacement, thus effectively creating a stiffer ground support (see Sect. 6.4.3.1).

### 6.4.3 Anchorage of Tendons

An understanding of the anchoring mechanism on which tendon performance depends provides insight into the applications and the limitations of tendons.

#### 6.4.3.1 Fully Encapsulated

Full encapsulation, or full column bonding, of a tendon in a drill hole provides one means of increasing the effective stiffness of a tendon support and reinforcement system. This is achieved using a polyester resin or a cementitious based grout. Although the encapsulating medium adheres to the tendon and to the drill hole wall, it is not a glue. Rather, it acts as an interference medium to generate shear resistance to the tendon being pulled out of the drill hole. In this text, the term **grout** is used generically to refer to all types of encapsulating medium.

Resin grouts comprise a catalyst and a filler (or mastic), which can be supplied in a two compartment cartridge that is inserted in the hole ahead of the tendon. The spinning action of the tendon breaks the cartridge and mixes the grout in the hole. Alternatively, the two components can be mixed as they are about to be pumped into the hole. Cartridge based grout systems may result in the last several hundred millimetres of the tendon remaining ungrouted at the collar of the drill hole, which is one reason for integrating a load bearing face plate into these systems.

A major advantage of resin grouts is that they can be designed to set within seconds and cure within hours, thereby providing immediate support. On the other hand, while cementitious grouts are slower to set and cure, they shrink less and provide better corrosion protection. Cementitious grout is usually pumped into a hole after the tendon has been placed. Resin may be used to end anchor a tendon prior to pretensioning and/or grouting.

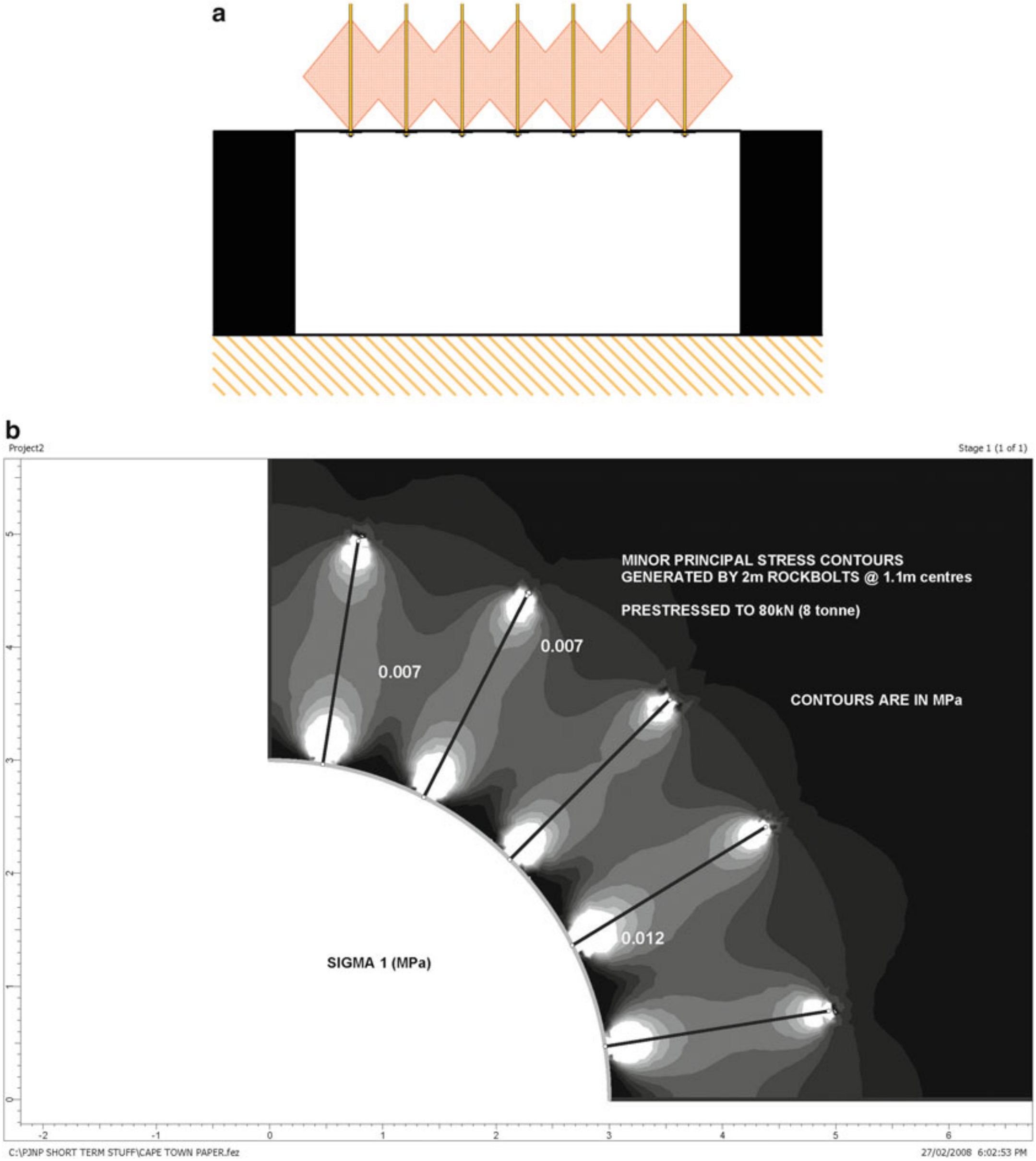
a
b
Project2
Stage 1 (1 of 1)
MINOR PRINCIPAL STRESS CONTOURS
GENERATED BY 2m ROCKBOLTS @ 1.1m centres
PRESTRESSED TO 80kN (8 tonne)
0.007
0.007
CONTOURS ARE IN MPa
0.012
SIGMA 1 (MPa)
0
1
2
3
4
5
-2
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**Fig. 6.34** Two diagrams used to show how tensioned tendons create a zone of compression within the reinforced horizon. (**a**) A conceptual diagram of a type often used to portray compressive stress induced by tensioned tendons, (**b**) Compressive stress magnitude and distribution as determined by numerical modelling (After Pells 2008)

In order for a parting plane to open, the rock mass on the free surface side of the parting has to slide past the tendon (Fig. 6.16). This induces shear stress in the encapsulation medium, which is transmitted to the tendon where it generates axial load. The converse then occurs on the opposite side of the parting. The axial load in the tendon generates shear stress in the grout, which is then transmitted back to the rock mass. This load transfer process generates a tensile force in the tendon, which acts to resist further opening of the parting plane. The clamping force increases with increasing displacement across the parting plane until the peak load capacity of one of the components of the reinforcement system is exceeded, these being the rock/grout interface, the grout, the grout/tendon interface, the tendon, and the tendon face plate and retaining hardware.

The length of grout anchor required for a tendon to reach its ultimate strength is a critical consideration, especially when using grout to only end anchor a tendon. Anchor capacity depends on many factors including the diameter of the tendon, the diameter of the drill hole, the surface profile and finish of the tendon, the surface profile of the drill hole, the cleanliness of the drill hole and the properties of the rock mass. In general, in soft and weak rock the critical strength parameter is the shear strength of the rock/grout interface and in strong rock it is the tendon/grout interface.

The surface profile of a tendon is important because it:

- increases shear resistance on the tendon/grout interface by creating mechanical interlock;
- increases shear resistance on both the tendon/grout and grout/rock interfaces by generating increased lateral confinement within the grout (and hence, normal stress on the interfaces) when shear movement occurs on the tendon/grout interface; and
- can significantly influence the effectiveness of the mixing process for two component resin capsules.

The manner in which lateral confinement is generated by differential movement between a solid bar and the encapsulating medium is shown in Fig. 6.35. This can be negated to some extent by a reduction in the bar diameter caused by elongation under load. This effect is more severe in cables because the strands have a tendency to untwist when placed under tensile load. Incorporating bulges, or bulbs, (referred to by a variety of names including ‘birdcages’, ‘ferrules’, ‘nuts’ and ‘Garforth bulbs’) into a cable at fixed intervals along its length counteracts this effect to some extent. The stiffness of these so called ‘modified’ cables increases with bulb density. The bulbs resist untwisting and generate higher levels of radial dilation, which increases shear resistance. Laboratory testing of 300 kN (30 t) capacity cables by Mosse-Robinson and Sharrock (2010) demonstrated how failure mode can be changed from frictional slip at the cable grout interface, to one of cable rupture by reducing the water to cement ratio of the grout and increasing bulb density. Hutchinson and Diederichs (1996) provide a detailed account of cable bolt anchorage mechanics.

In addition to the lateral confinement generated by differential movement between a

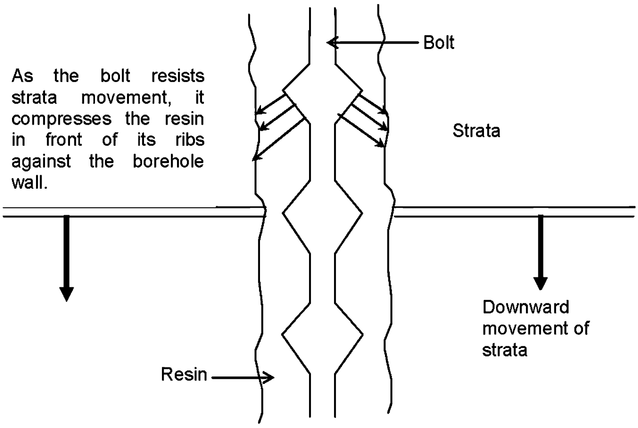

As the bolt resists strata movement, it compresses the resin in front of its ribs against the borehole wall.
Bolt
Strata
Downward movement of strata
Resin

**Fig. 6.35** Schematic depiction of how shear displacement on the grout/bar interface generates lateral confinement (Adapted from Offner 2000)

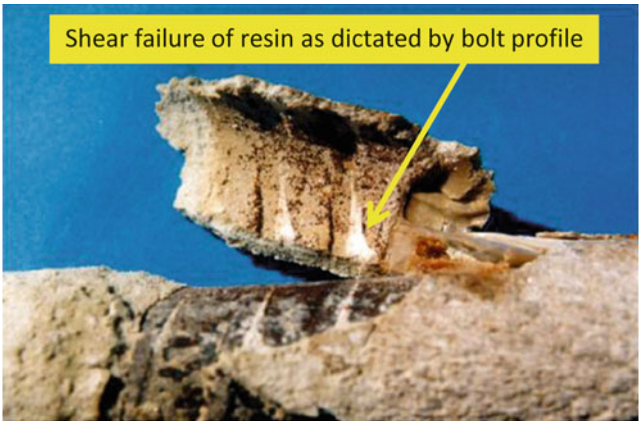


**Fig. 6.36** Shear failure pattern on grout/tendon interface associated with generation of lateral confinement (After Galvin et al. 2001a)

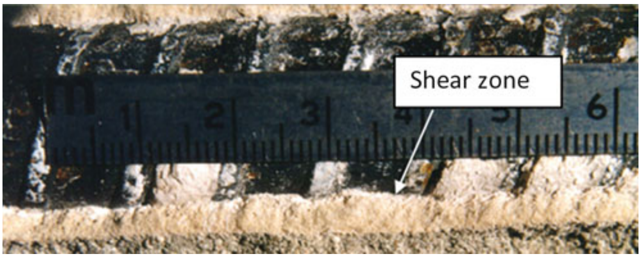


**Fig. 6.37** Narrow shear failure zone at grout/tendon interface after mechanical interlock has been overcome (After Galvin et al. 2001a)

tendon and the grout, the performance of the anchor also depends on the external confining stress acting on the drill hole at the time that the anchor is set. A mining induced relaxation in confining stress can result in a considerable reduction in anchorage capacity as a result of slip at the grout/rock interface (Kaiser et al. 1992; Offner 2000; Moosavi et al. 2002).

Figure 6.36 shows the appearance of the shear failure pattern on the grout/tendon interface after a bolt was pull tested to 20 mm in the UNSW Rockbolt Test Rig and then extracted from the core. The variable width of the shear zone is a direct function of the geometry of the tendon profile. The final shear profile plane is a function of the profile height, while the failure of the (resin) grout to this height is dictated by the angle of the profile. Figure 6.37 shows the narrow shear failure surface that developed in the resin at the top of the bolt profile after adhesion and mechanical interlock had been overcome. The residual load transfer capacity of the anchor is determined by the frictional, or mechanical interlock, properties of this interface.

The shear force, $f_a$, mobilised per unit length of cable as a result of relative shear displacement, $u_a$, between a tendon surface and the walls of a borehole is related to the stiffness of the encapsulating medium, $k_{bond}$. This stiffness is given by Eq. 6.5 (Brady and Brown 2006).

$$k_{bond} = \frac{2\pi G_e}{ln\left(1 + \frac{2t}{d_T}\right)} \tag{6.5}$$

where

$G_e = shear\ modulus\ of\ encapsulating\ medium$
$t = grout\ annulus\ thickness$
$d_T = diameter\ of\ the\ tendon$

The formula shows that the stiffness of the grout bond increases as annulus thickness decreases for a given diameter tendon, or as tendon diameter increases for a given annulus thickness. A range of literature based on analyses, laboratory studies and field tests reports that annulus values in the range of 2–7 mm result in the best load transfer (e.g. Franklin and

Woodfield 1971; Fairhurst and Singh 1974; Dunham 1976; Ulrich et al. 1989; Wagner 1995; Campoli et al. 1999; Wilkinson and Canbulat 2005; Spearing et al. 2011).

Fabjanczyk and Tarrant (1992) concluded on the basis of laboratory push tests (as distinct from the standard pull test) on annulus sizes down to 1.5 mm that the optimum annulus size is the smallest which can be used given practical constraints. Fuller and O'Grady (1994) reported that Flexibolt cables installed in 28 mm holes had a 40 % higher peak pull-out load and a 30 % higher pull-out stiffness than when installed in 29 mm diameter holes. These results contrast with the outcomes of laboratory studies by Aziz (2004) which found that hole diameters up to at least 35 mm made no difference to the performance of bolts with a nominal diameter of 21.7 mm when the encapsulating resin was pre-mixed. However, performance did decrease with increasing hole diameter when resin cartridges were employed, which Aziz (2004) attributed to the degree of gloving (see Sect. 6.4.4) and poorer mixing of the resin. Aziz's findings fall at the upper bound of the optimum annulus range noted previously. ACARP (2014a) reported that research in progress had concluded that bolts installed in 27 mm diameter holes performed better than those installed in holes larger than 28 mm.

Another contributory factor to anchor performance is the viscosity of the encapsulation medium and the pressure front that this can create at the back of the hole as a grout capsule is pushed ahead of the tendon. While Eq. 6.5 indicates that shear resistance should increase as annulus thickness is reduced, it appears that the quality of the anchor installation is jeopardised in small annulus situations due to practical difficulties associated with forcing the encapsulation medium into a narrow annulus. The pressures generated towards the back of the drill hole can be so great that the host rock is fractured and the encapsulating medium is forced into these fractures and other partings.

It is Australian underground coal mining practice to install standard rock bolts with a nominal diameter of 21.7 mm in 27 mm or 28 mm diameter holes drilled with wet flushing. Drill bit size may be increased up to 32 mm in the presence of reactive clay bands in order to overcome hole closure problems during drilling and tendon installation. However, the larger diameter results in reduced anchorage capacity, which operators often attribute to ineffective mixing of the resin due to the larger annulus. Some mines have reverted to dry drilling in these situations, in which case flushing of the holes to remove dust has proven very important to achieving good anchorage. Anchorage may be improved in weak strata by increasing the degree of rifling of the drill hole walls or by using a larger diameter drill hole to increase the rock/grout contact area. If hole diameter is increased, it follows that tendon diameter may also need to be increased in order to maintain load transfer capacity across the grout annulus.

Pull testing by attempting to jack a tendon out of a drill hole does not permit the support capabilities of a tendon system to be determined, other than for end anchored systems. In the case of fully encapsulated tendons, only a short length of the system might be tested before the bolt breaks, providing no idea of anchorage characteristics further along the drill hole. Therefore, the load transfer capacity and characteristics of a grout anchor are tested by conducting pull-out tests on short embedment lengths. This usually involves inserting a hollow hydraulic cylinder over the protruding end of the tendon and applying tension to it by reacting against the collar of the drill hole. The highest shear stress on the tendon is therefore generated at its free end and dissipates down its length. This is the converse of what occurs in fully encapsulated tendons in the field, where dilation at a parting plane creates two free ends that are subjected to the highest shear stress, with this stress dissipating towards the front and back of the drill hole.

Short encapsulation pull tests by Hawkes and Evans (1951) and Farmer (1975) showed that the distribution of shear stress along a grouted bolt in a rigid socket can be described by an exponential function. Finite element analysis by Coates and Yu (1970) confirmed this behaviour for moderate to high ratios of tendon modulus to rock modulus. However, for very soft rock, the shear stress was almost uniformly distributed along the length of the bolt. Since the shear stress at the bolt/grout interface is linked to the axial stress, it follows that the axial stress will decrease in a similar manner. Signer (1990) undertook field testing utilising strained gauged bolts in a coal mine setting and also produced an exponential axial load transfer distribution for fully encapsulated tendons, with the slope of the load transfer lines progressively becoming flatter with increase in applied load (Fig. 6.38). Wade et al. (1977) reported a similar finding.

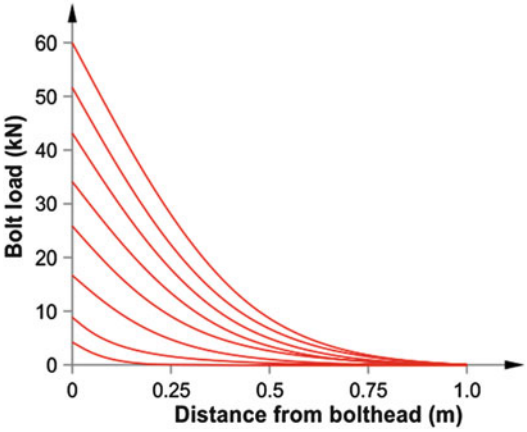


**Fig. 6.38** Average field test results from measuring load decay along instrumented bolts (Adapted from Signer 1990)

Farmer (1975) developed an analytical solution for the approximate shear stress distribution along a typical steel/resin anchor which, when applied to a 20 mm diameter tendon installed in a 27 mm diameter hole, has the form given by Eq. 6.6. The resulting stress distribution is plotted in Fig. 6.39.

$$\frac{\tau(x)}{\sigma_o} = 0.1\ e^{\left(-0.36\frac{x}{d_T}\right)} \tag{6.6}$$

where

$x =$ *distance along bolt measured from the free end*
$\tau(x) =$ *shear stress at distance x*
$\sigma_o =$ *axial stress in the bolt*
$d_T =$ *tendon diameter*

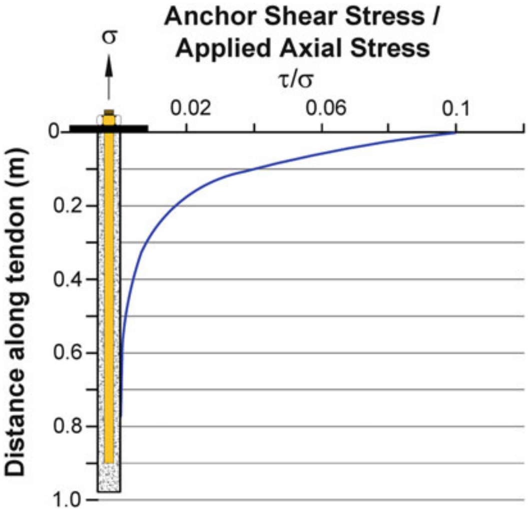


**Fig. 6.39** Theoretical shear stress distribution along a fully encapsulated tendon in a rigid socket with a thin resin annulus based on the formulation of Farmer (1975)

An alternative model of a linear load transfer distribution within a fully encapsulated tendon has also been postulated. It has two forms, one with load reducing linearly to zero at some point along the tendon and the other with load reducing linearly to zero at the back end of the tendon. The latter does not appear to be supported by measured field data or numerical modelling and is incompatible with basic Newtonian physics. However, there is a range of field measurements and numerical modelling outcomes that are not inconsistent with the former. Aspects of this model have been researched by Whitaker (1998), Offner (2000), Galvin et al. (2001a, b) and Hagan (2003). Three of the more important contributing factors to the difference between laboratory outcomes and those measured in the field appear to be:

- the difference in loading regime in the field, whereby a tendon is loaded in the field by dilation at parting planes within the rock mass rather than by a pulling force which reacts against the collar of the tendon;
- field tendons are considerably longer; and

- field forces are much higher than those applied in the laboratory.

From a practical perspective, it appears that the differences between the two models are not significant. The measured and computed load distribution profiles shown in Figs. 6.38 and 6.40, for example, display a rate of exponential decay that can be approximated to a linear rate of decay without introducing significant error. This is not inconsistent with NIOSH cable bolt testing outcomes reported by Martin et al. (2004).

In areas where the shear stress reaches the shear strength of the grout, local debonding will occur and cause the maximum sustainable shear stress to migrate further along the bolt (Wagner 1995). Field experience has shown that pull-out resistance, or load transfer, for a 20 mm diameter rock bolt grouted in sedimentary rock is typically in the range of 300–600 kN/m (30–60 t/m) but may be as high as 1.1 MN/m (110 t/m).

In order to better conceptualise how encapsulated tendons respond to dilation on parting planes, load transfer profiles can be approximated to triangular distributions, with rock mass conditions assumed to be uniform on either side of the parting planes (Galvin and Wagner 1994). This approach has been applied in Figs. 6.41, 6.42, 6.43, 6.44, 6.45, and 6.46. No allowance has been made for any contribution that lateral shear may make to axial load. Furthermore, the grout component of the systems has been assumed, for the moment, to have an indefinite load transfer capacity. In reality, tendon-grout and grout-rock interfaces cannot accept load transfer indefinitely as assumed in the models and ultimately decoupling (debonding and slippage) develops at the parting plane and starts to work its way along the tendon. A reduction in the diameter of the tendon as it is stretched under load can contribute to this decoupling. Residual shear strength prevents an immediate and total loss of support resistance in the decoupled section.

Figure 6.41a shows a parting plane that has developed some distance along the length of a tendon. The tensile force generated in a tendon by a given amount of displacement, Δl, across the parting plane is directly proportional to the stiffness of tendon. In accordance with Eq. 2.3, tendon stiffness increases with increasing modulus and cross sectional area of the tendon and decreasing length of tendon subjected to the force. The clamping force, $L_T$, generated in the tendon is given by Eq. 6.7.

$$L_T = \Delta l\, k_T = \Delta l\, E_T \frac{(d_T)^2 \pi}{4 l_{LT}} \tag{6.7}$$

where

$L_T$ = *force generated in tendon*
$\Delta l$ = *total dilation across parting plane*
$k_T$ = *tendon* stiffness
$E_T$ = *elastic modulus of tendon material*
$d_T$ = *tendon diameter*
$l_{LT}$ = *total load transfer distance*

When shear resistance along the tendon/grout interface is high, the length of tendon over which load transfer takes place, $l_{LT}$ , is short. This is reflected in the steep gradient of load transfer line '1' plotted in Fig. 6.41b. Therefore, the local stiffness in the vicinity of a parting is high and so the tendon rapidly builds up load, $L_{T1}$, in response to dilation across the parting plane.

Load transfer line '2' has a flatter gradient that reflects a lower rate of load transfer, resulting in a softer support system. The peak load carrying capacity for the geometry depicted is reached when the load transfer curve extends to the back end of the tendon (Fig. 6.41c). At that point, the tendon will start to be pulled out of the rock mass if there is no reduction in the tendon load across the parting plane. If the tendon had not been fitted with a face plate and retaining system (of designated load carrying capacity, $L_{P\ peak}$) the system would have failed earlier by the rock mass sliding off the front of the bolt.

In the case of the higher load transfer environment, depicted by curve '1', ongoing dilation ultimately results in the load in the tendon across the parting reaching the yield strength of tendon, $L_{T\ yield}$, as shown in Fig. 6.42c.

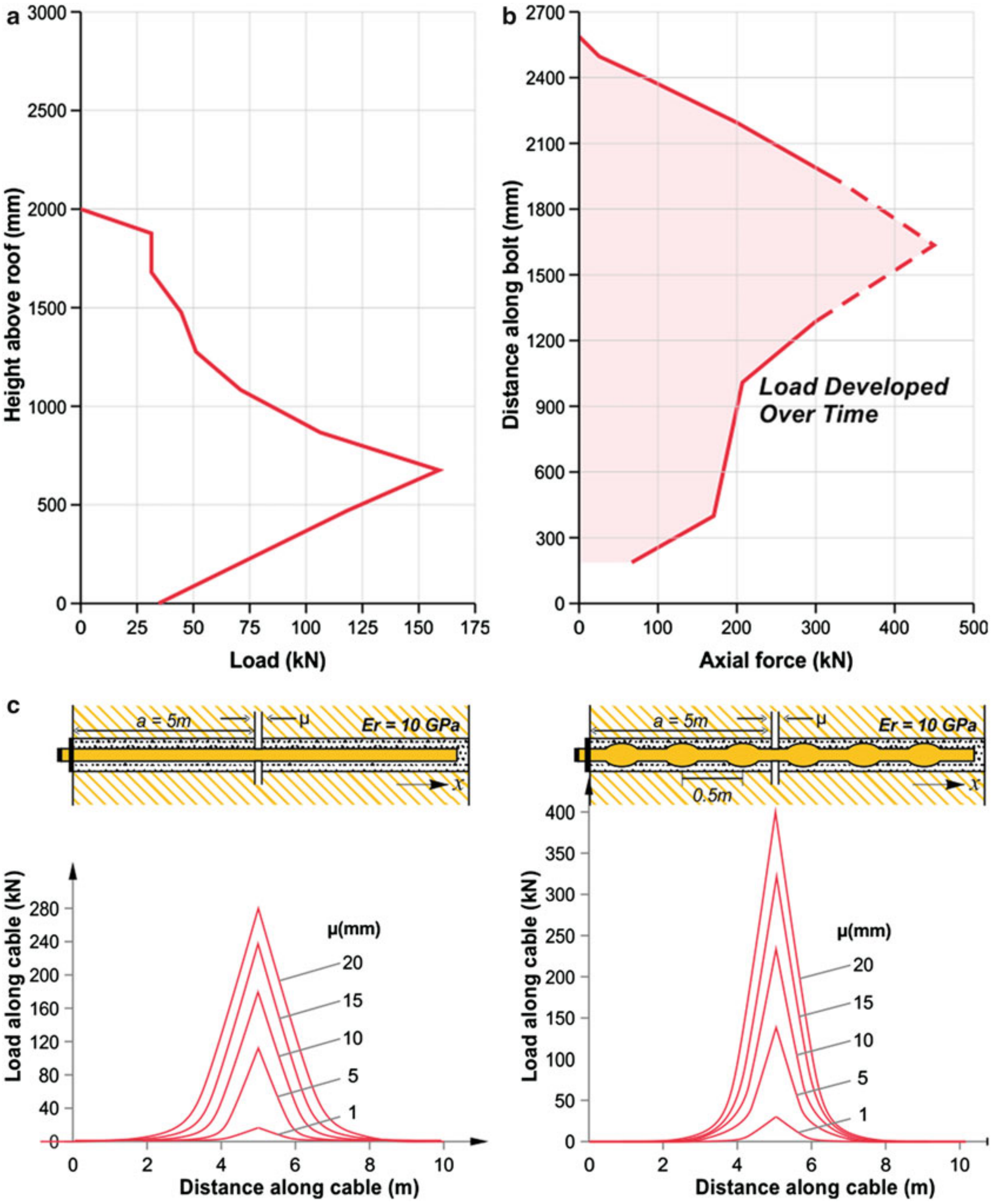


**Fig. 6.40** Examples of load distributions in fully encapsulated tendons as determined using strain gauged tendons in the field (**a**, **b**) and numerical modelling (**c**). (**a**) Field measurements, (**b**) Field measurements (After Gray et al. 1998), (**c**) Numerical modelling (After Moosavi et al. 2002)

The location of a parting plane has an important influence on the failure mode of a fully encapsulated tendon. Figure 6.43 illustrates that if the parting plane shown in Fig. 6.42 were located closer to the free surface, the system would fail due to the load bearing capacity of the face plate assembly being exceeded. Conversely, if it was located closer to the back of the tendon, the peak capacity of the system would be determined by anchor slippage at the

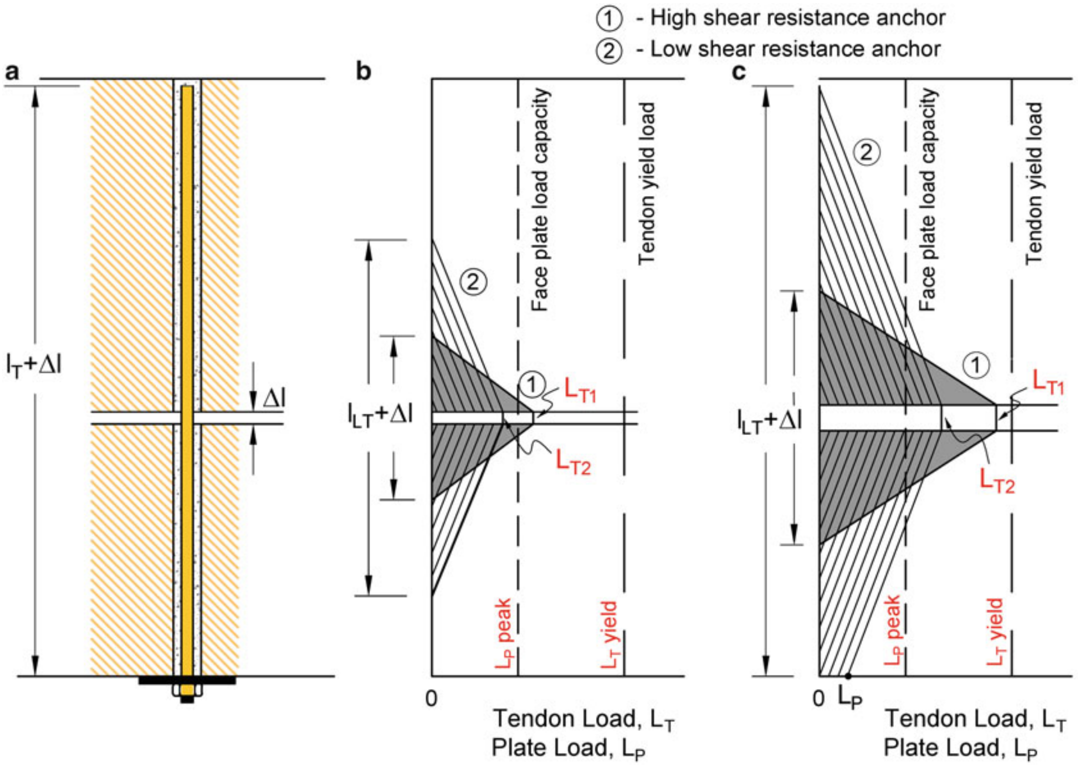

① - High shear resistance anchor
② - Low shear resistance anchor
a
b
c
$l_T+\Delta l$
$\Delta l$
$l_{LT}+\Delta l$
Face plate load capacity
Tendon yield load
②
①
$L_{T1}$
$L_{T2}$
$L_P$ peak
$L_T$ yield
0
$L_P$
Tendon Load, $L_T$
Plate Load, $L_P$

**Fig. 6.41** Influence of shear stiffness of anchor system on load transfer in a fully encapsulated tendon. (**a**) Loading location, (**b**) Intermediate state, (**c**) Tendon 2 fails first, due to anchor slippage at back of hole

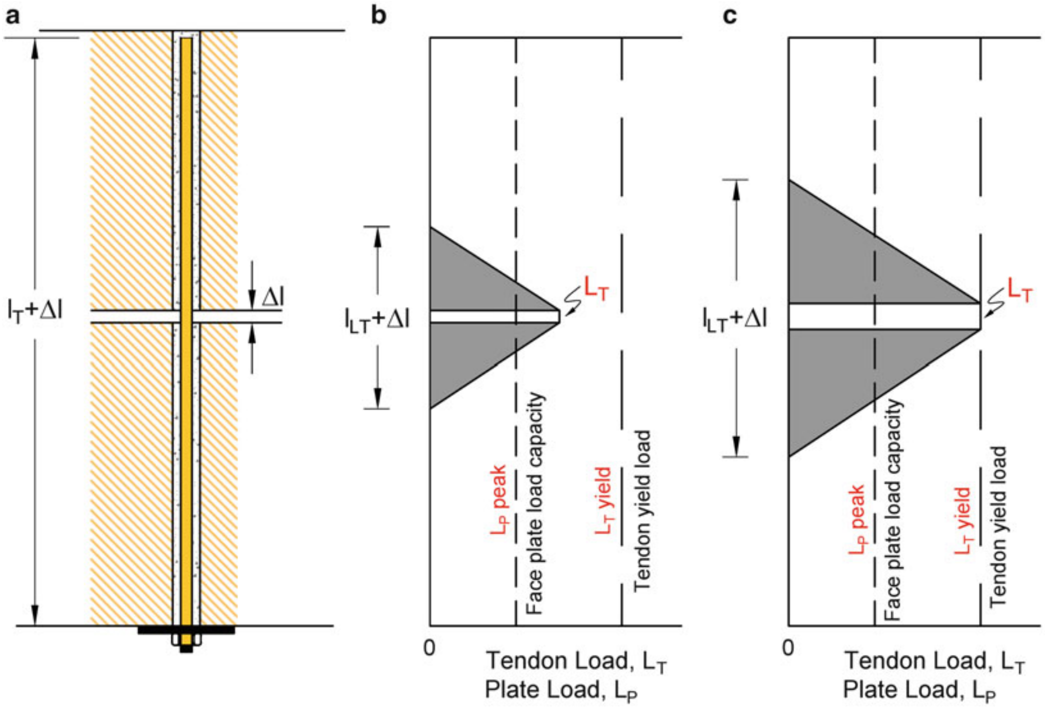

a
b
c
$l_T+\Delta l$
$\Delta l$
$l_{LT}+\Delta l$
$L_T$
$L_P$ peak
Face plate load capacity
$L_T$ yield
Tendon yield load
0
Tendon Load, $L_T$
Plate Load, $L_P$

**Fig. 6.42** Load transfer diagrams associated with axial yield of a fully encapsulated tendon. (**a**) Loading location, (**b**) Intermediate state, (**c**) Failure due to exceeding yield strength of tendon

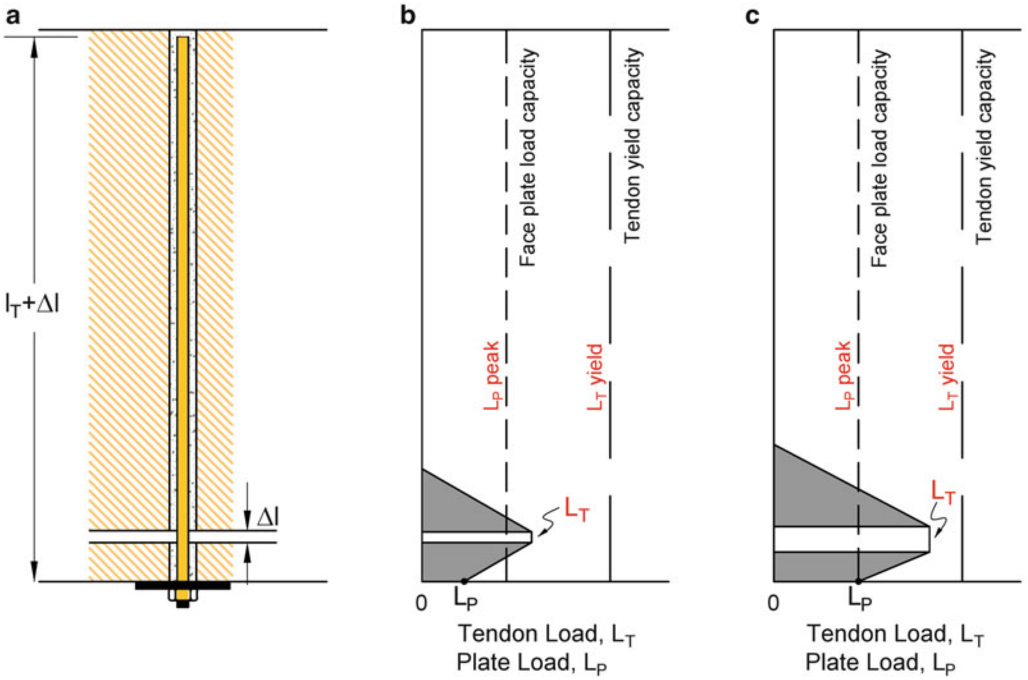

a
$l_T+\Delta l$
$\Delta l$
b
Face plate load capacity
Tendon yield capacity
$L_P$ peak
$L_T$ yield
$L_T$
0
$L_P$
Tendon Load, $L_T$
Plate Load, $L_P$
c
Face plate load capacity
Tendon yield capacity
$L_P$ peak
$L_T$ yield
$L_T$
0
$L_P$
Tendon Load, $L_T$
Plate Load, $L_P$

**Fig. 6.43** Load transfer diagrams associated with loss of anchor shear resistance capacity at the front of a tendon and consequential load transfer leading to failure of the face plate system. (**a**) Loading location, (**b**) Intermediate state, (**c**) Failure due to exceeding load capacity of face plate

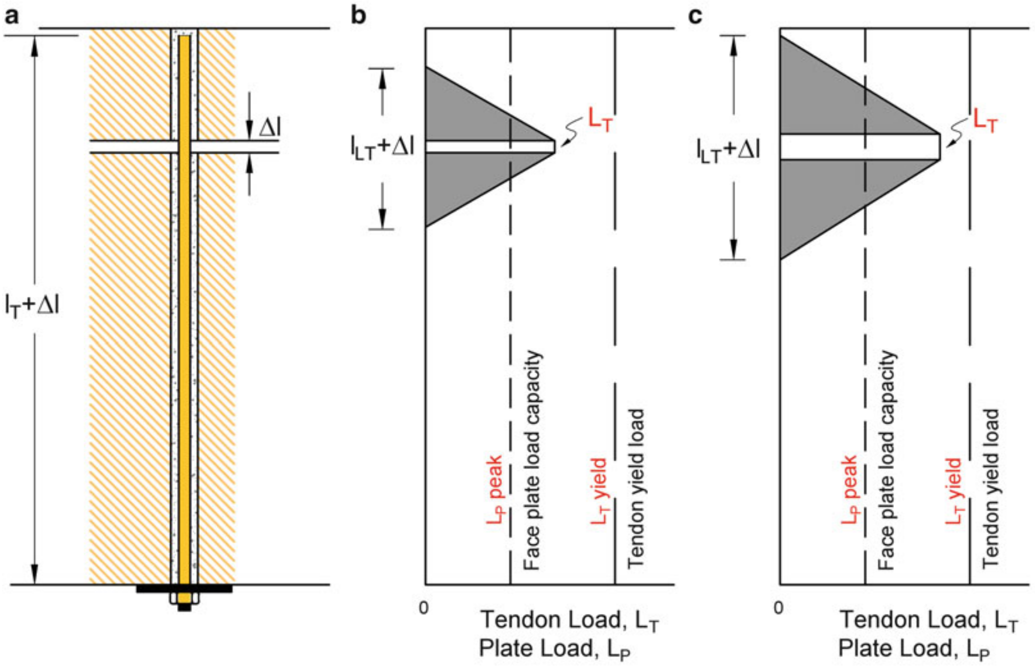

a
$\Delta l$
$l_T+\Delta l$
b
$l_{LT}+\Delta l$
$L_T$
$L_P$ peak
Face plate load capacity
$L_T$ yield
Tendon yield load
0
Tendon Load, $L_T$
Plate Load, $L_P$
c
$l_{LT}+\Delta l$
$L_T$
$L_P$ peak
Face plate load capacity
$L_T$ yield
Tendon yield load
0
Tendon Load, $L_T$
Plate Load, $L_P$

**Fig. 6.44** Load transfer diagrams associated with loss of anchor shear resistance capacity at the back of a tendon. (**a**) Loading location, (**b**) Intermediate state, (**c**) Failure due to anchor slippage at back of hole

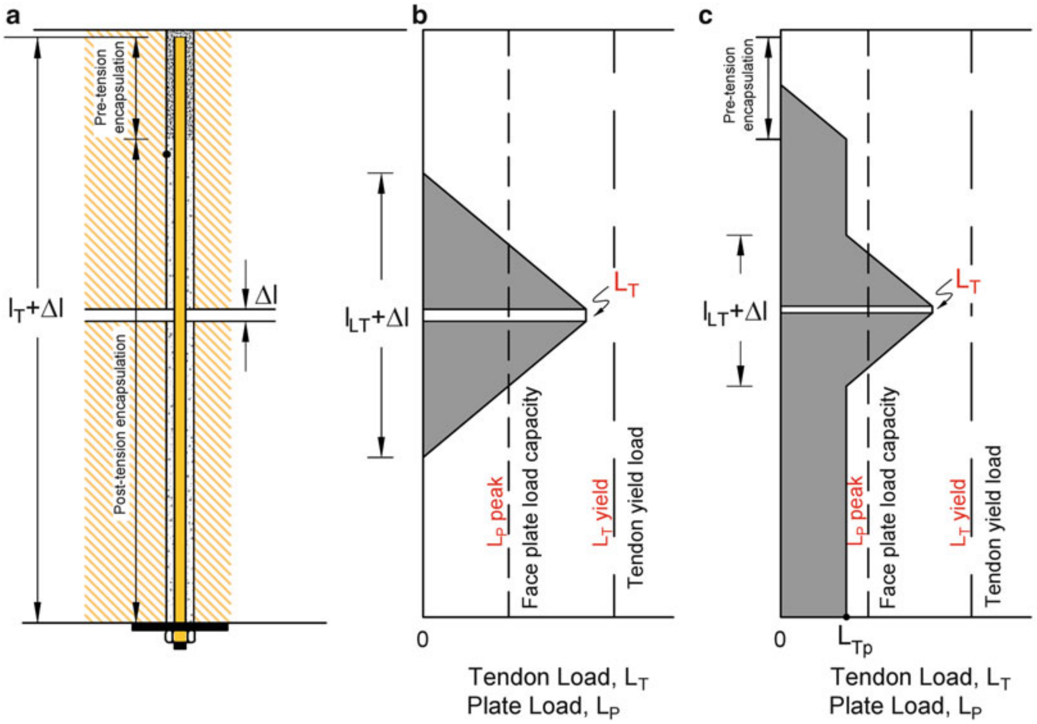
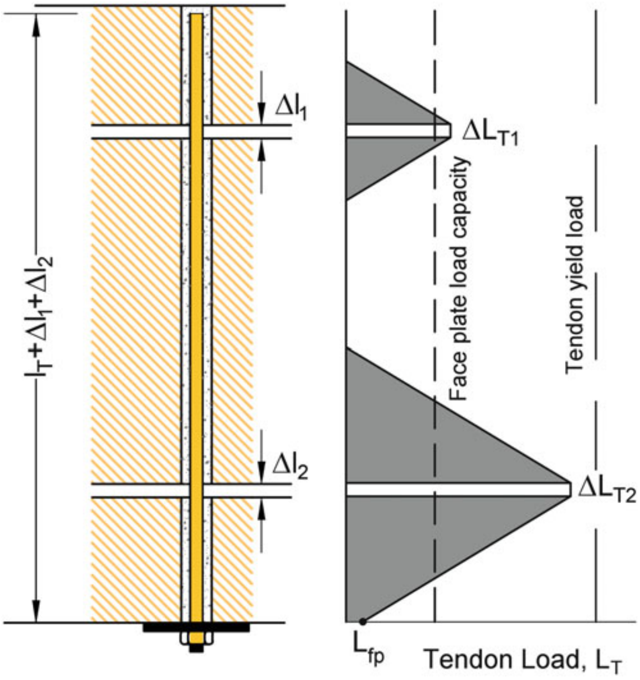

a
b
c
Pre-tension encapsulation
Post-tension encapsulation
$l_T+\Delta l$
$\Delta l$
$l_{LT}+\Delta l$
$L_T$
$L_P$ peak
Face plate load capacity
$L_T$ yield
Tendon yield load
0
$L_{Tp}$
Tendon Load, $L_T$
Plate Load, $L_P$

**Fig. 6.45** Load transfer diagrams associated with a pretensioned, fully encapsulated tendon. (**a**) Loading location, (**b**) Tendon not pre-tensioned, (**c**) Tendon pre-tensioned

**Fig. 6.46** Load transfer diagrams associated with multiple parting planes along a fully encapsulated tendon


$l_T+\Delta l_1+\Delta l_2$
$\Delta l_1$
$\Delta l_2$
$\Delta L_{T1}$
$\Delta L_{T2}$
Face plate load capacity
Tendon yield load
$L_{fp}$
Tendon Load, $L_T$

back of the hole (Fig. 6.44). In both cases, the maximum load bearing capacity of the tendon is not utilised.

In some cases, a highly stressed or failed face plate assembly has been attributed to poor load transfer arising from factors such as sub-standard installation technique or incomplete encapsulation due to loss of grout in fractures. However, as Fig. 6.43 demonstrates, load transfer could in fact be up to standard, with the presence of a parting plane towards the front of the hole accounting for the high collar loading. The installation of extensometers on a regular basis aids in establishing the correct mechanism of tendon behaviour.

Figure 6.45c illustrates how applying a pretension results in a higher tendon reaction load for any given amount of displacement across a parting plane. Hence, pretensioning is another means of increasing the effective stiffness of a tendon support and reinforcement system. However, this benefit is reduced or lost if a parting plane develops towards the back or the front of the tendon. This is because the parting may only have to open a small amount to result in slippage of the already preloaded point anchor (if the parting is towards the back of the hole) or failure of the pretensioned face plate components (if the parting is towards the front of the hole).

It follows from these basic models that a fully encapsulated tendon has the potential to generate significant clamping forces at multiple locations along its length, as shown in Fig. 6.46, with ultimate load capacity not necessarily being determined by cumulative displacement over the length of the tendon. The opportunity to exploit these benefits increases with increase in tendon length.

Strain gauged bolts and drill hole extensometers provide field evidence of the behaviours depicted in the preceding simple models. However, the stress distributions are usually more complex and irregular than depicted in these models because the tendon-grout and grout-rock interfaces do not accept load transfer indefinitely as assumed in the models, rock mass conditions vary, and multiple parting planes develop over the length of a tendon.

The rapid rate of face advance in coal mining favours the use of resin grouts for primary support systems because of their rapid set and short curing times. Full column pretensioning may be achieved with a resin anchorage system by placing a fast set resin at the back of the hole and a slow set resin in the remainder of the hole; thereby providing sufficient time to tension the bolt in between the setting of the two mixtures. Secondary support systems favour the use of cementitious grout, although a fast set resin or mechanical anchor may be used to initially secure a tendon in place and permit it to be tensioned prior to full column cementitious grouting.

Full column bonding offers increased resistance to rock mass bedding plane shear and increased corrosion protection. When grout is metered in fixed quantities, such as in cartridges, it is important that holes are not over-drilled as this results in grout being consumed by the void at the end of the hole. When resin grout is introduced into the hole in cartridges, the load transfer between the tendon and the rock mass can be adversely impacted by ‘gloving’ (discussed in Sect. 6.4.4.2).

In summary, it is important to be aware when utilising fully encapsulated tendons that:

- A tendon may exhibit few signs, if any, at its collar that it is under a high state of stress, or that it has broken, or that it is slipping due to shear failure of the anchor at the back of the hole.
- A tendon can generate high support reactions simultaneously at multiple parting planes.
- A broken tendon can retain a high capacity to transfer load across other parting planes remote from the break.
- The capacity of the face plate assembly can be very important to achieving the load transfer capacity of a fully encapsulated tendon, especially when partings are located towards the front of the tendon.

- The use of pretensioned tendons may require a balance to be struck between generating high initial system stiffness to resist displacement while also retaining adequate capacity to tolerate displacement.
- Monitoring, particularly extensometry, is important for developing a proper understanding of tendon behaviour and standard of performance.

### 6.4.3.2 End Anchored Tendons

An end anchored tendon is fixed at the back of a drill hole by means of either a short embedment length of grout or a mechanical anchor and is fitted with a face plate and retaining mechanisms at the collar of the hole. Some older mines may still have areas supported by slot and wedge mechanical anchors. The anchorage principles that apply to fully encapsulated tendons also apply to grouted end anchors.

An end anchored tendon is a much softer system than a fully encapsulated tendon, all other things being constant. This is because the stretch induced in an end anchored tendon by a given amount of rock dilation is distributed over its full length rather than concentrated over a portion of its length. Therefore, a lower reaction force that is distributed uniformly along the length of the tendon is generated (Fig. 6.47). The reaction force decreases with increase in tendon length. This has important implications when using long, end anchored cables that are not subsequently fully encapsulated, since the cable may allow excessive dilation and loss of ground control well before its ultimate load bearing capacity is reached. Installing two cables per hole to increase effective tendon diameter and, therefore, tendon stiffness is one means of addressing this behaviour. On the other hand, in some circumstances, the lower stiffness of a long tendon that is only end anchored can be utilised to advantage to provide a controlled rate of convergence (see Sect. 7.5.3).

Most mechanical anchors operate on a barrel and wedge principle, whereby a central barrel forces surrounding wedges to expand laterally over a short length of the hole as the head of the tendon is rotated or stretched and tightened against the face plate of the tendon (Fig. 6.47). The wedges, or shell, can be held in place with a nut that causes the back of the wedges to bite into the borehole, or with a spring steel strap (referred to as a 'bail arm') that results in the wedges making uniform contact with the borehole walls. The former is claimed to be better suited to hard rock and the latter to weak rock. The anchoring arrangement can generate high contact stresses on both the walls of the drill hole at the back of the hole and beneath the face plate at the collar of the drill hole.

The load bearing capacity of an anchor is determined by the magnitude of the drill hole contact stresses, which can be of the order of 20–30 MPa (Wagner 1985a) and the contact and shear strength of the rock in which it is located. The load generated by the face plate is a function of the contact strength of the rock that it reacts against and the deformation properties of the plate and associated retaining hardware.

Mechanical end anchored tendons have the following characteristics:

- They are relatively cheap, easy to install, tolerant of minor variations in hole diameter and insensitive to over-drilling of the hole length.
- The anchor mechanism operates in a manner such that an increase in tendon load causes an increase in anchor contact stress. This can be beneficial in strong or hard ground and a disadvantage in weak or soft ground.
- Provided that the anchor is well set, a high load can be suspended from a short bolt.
- They are prone to lose tension over time due to creep and localised rock fracturing induced by high contact stresses at the anchor or the face plate of the tendon.
- They are prone to lose tension when subjected to blast vibrations.
- Because the drill hole diameter is usually considerably greater than the bolt diameter

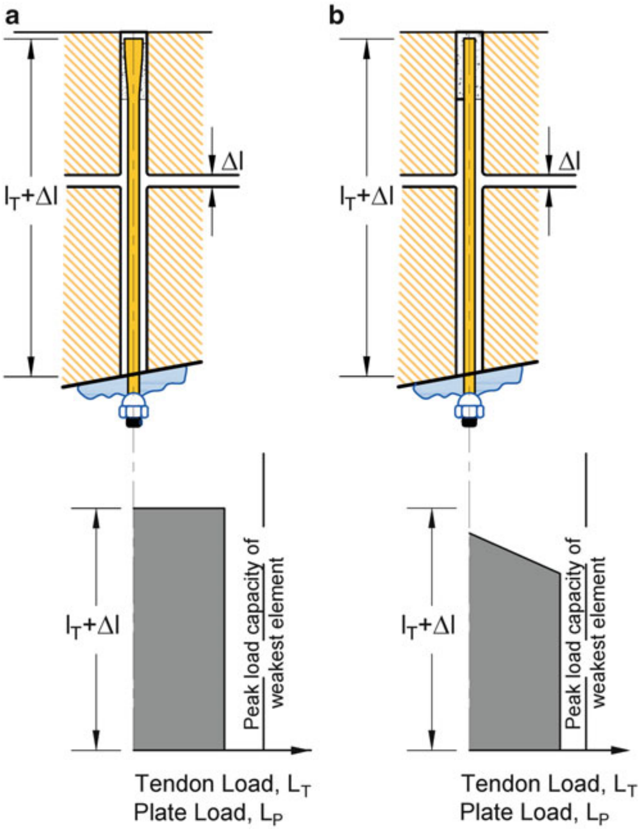

a
b
$l_T+\Delta l$
$\Delta l$
$l_T+\Delta l$
$\Delta l$
$l_T+\Delta l$
Peak load capacity of weakest element
Tendon Load, $L_T$
Plate Load, $L_P$
$l_T+\Delta l$
Peak load capacity of weakest element
Tendon Load, $L_T$
Plate Load, $L_P$

**Fig. 6.47** Load transfer diagrams associated with end anchored tendons. (**a**) Mechanically end anchored, (**b**) Grout end anchored

(in order to fit the anchor), end anchored tendons offer limited initial resistance to shear displacement on bedding and fracture planes and are less effective than fully encapsulated bolts in stabilising stratified rock.

- The lack of full encapsulation makes them more vulnerable to corrosion.

Hence, mechanically end anchored tendons are more suited to suspending loose slabs of rock over limited distances, with Wagner (1995) advising that they should be anchored in rock that has a uniaxial compressive strength of more than 50 MPa.

Grout end anchored tendons are less susceptible to loss of tension over time. However, they are more prone to slip in bore holes that have smooth and/or greasy sides, such as in the case shown in Fig. 6.48, and their effectiveness is more sensitive to the installation procedure and to the length of encapsulation. Both mechanical and grout end anchored tendons:

- Should have their face plates tightened against a flat surface. This may be facilitated by the use of a hemispherical washer between the plate and the head of the tendon.
- Depend on the load carrying capacity of the face plate and its restraining system.
- May need to be re-tensioned periodically.

#### 6.4.3.3 Friction Anchored Tendons

The majority of friction anchored tendons employed in underground coal mining comprise either:

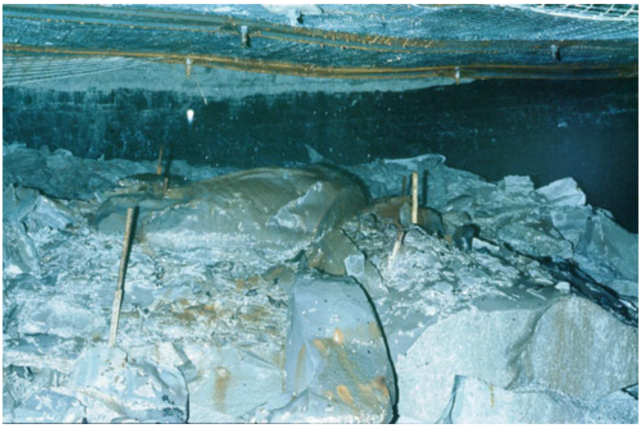


**Fig. 6.48** A roof fall associated with slippage of resin end anchors at the resin/rock interface

- a split steel tube referred to as a Split Set® of typically 38 mm diameter which is impact driven into a drill hole some 3 mm smaller in diameter; or
- a steel tube folded into an omega (Ω) shape and sealed at both ends and referred to as a Swellex® bolt, which is inserted into a drill hole of typically 38 mm diameter and expanded using water pressure.

The radial force exerted by a tube generates frictional resistance to sliding of dilating rock on the steel (Fig. 6.17). Rusting of the outer surface of the tube increases this resistance. As with grouted anchors, the length of anchor contact with the rock mass is critical to the tendon achieving its ultimate strength.

Characteristics of friction anchored tendons include:

- they are easy to install;
- they provide immediate support;
- they can tolerate a large amount of shear;
- the anchor holding force changes little with slippage;
- anchorage capacity is very sensitive to drill hole diameter;
- they have a considerably lower anchorage capacity than grouted anchors;
- their performance is not affected by over-drilling of the holes;
- they are susceptible to corrosion because of their high exposed internal surface area. Post grouting reduces this susceptibility but may not eliminate it, especially in an acid mine water environment; and
- they cannot be pretensioned.

## 6.4.4 Practical Considerations

### 6.4.4.1 Face Plate Assemblies

A face plate assembly comprises a load bearing plate that fits through the end of a tendon and a washer and nut or a barrel and wedge for retaining and tightening the plate against the rock face (Fig. 6.49). If the roof profile is irregular or the tendon is not installed normal to the rock face, the assembly may also include some form of spherical seat to enable the plate to sit flush against the rock face; to prevent it from being point loaded by the retaining mechanism; and to avoid inducing high bending moments in the tendon. Some face plate assemblies are purpose designed for the intended angle of installation. However, these find limited application in underground coal mining since, because of the large number of tendons installed in this environment, there is an increased likelihood of incorrect installation.

Ideally, the capacity of the face plate assembly should at least match the ultimate strength of the tendon. However, this is not always achievable, especially if the surface rock is uneven or weak or if the tendon has a very high load capacity. A face plate assembly may also be utilised to impart a yielding capacity to the tendon. This is

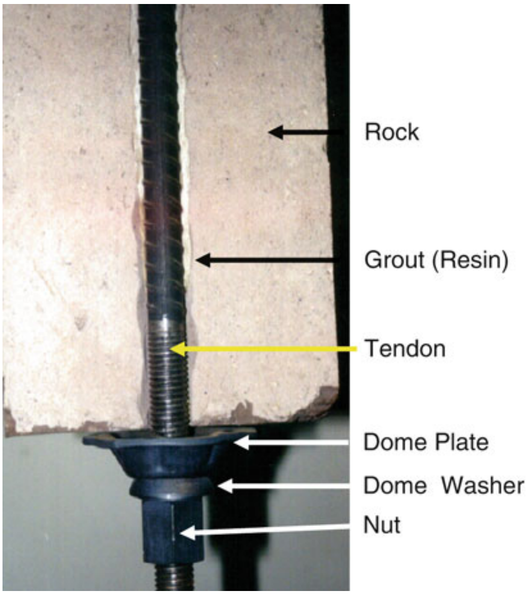


**Fig. 6.49** Components of a simple face plate assembly

often achieved by using some form of dome-shaped, spring bearing plate that progressively flattens under increasing load, or by changing the composition of the bearing plate (for example, from steel to timber). As a point of reference, a standard 10 mm thick, 150 mm square, flat steel plate has a load capacity of about 300 kN (30 t), while that of so-called 'cup and saucer' configurations can be up to 800 kN (80 t).

#### 6.4.4.2 Gloving

The term **gloving** refers to the plastic casing of a resin cartridge (capsule) partially or completely encasing a length of tendon, typically with a combination of mixed and unmixed resin filler (mastic) and catalyst remaining within the cartridge (Campbell et al. 2004). Typical examples of gloved roof bolts and unmixed chemicals are shown in Fig. 6.50.

Gloving is an important consideration in all circumstances but especially when an horizon towards the back of a drill hole is targeted specifically for the anchoring of reinforcement. At present, the only means to detect gloving is by overcoring, which is a slow, costly and destructive process.

The occurrence of gloving has been a concern since the widespread introduction of resin anchor cartridges in the 1980s. Research by Campbell and Mould (2003) and Campbell et al. (2004) revealed that the problem was widespread across resin brands, roof types, installation methods, collieries and countries and could not be attributed to poor installation practice. It concluded that the mechanism involves the development of a pressure front that could exceed 4 MPa as the bolt encountered the resin cartridge, causing the cartridge to expand radially so that the bolt could then be spun inside the cartridge. This results in insufficient contact interference to properly shred the cartridge and poor mixing of the mastic and catalyst components.

In the case of gloving associated with 900–1,000 mm long cartridges, 70 % of the recovered gloved bolts had an unmixed length in excess of 400 mm. The high pressure front also forced the resin into surrounding fractures, causing severe loss in the volume of resin on some occasions. An instrumented bolt installed under standard field conditions confirmed the lack of load transfer over a 400 mm long gloved section at the back of the hole, where the resin components were not adequately mixed (Fig. 6.51).

Research into this problem has included studies of tendon type, hole length and the formulation of the resin cartridges. Trials of various modified bolt profiles concluded that a chamfered bolt end, a wriggle (sinuous) bolt shank and an off-centre nut produced the best results, achieving good encapsulation over a distance in excess of 90 % of the effective bolt length (Campbell et al. 2004).

Laboratory tests in the USA and Australia indicated that load transfer was not reduced significantly in a gloved section where the resin had been properly mixed and set. Compton and Oyler (2005) suggested that the low proportion of catalyst in Australian resin cartridges (being typically 6–7 %) in comparison to that in USA resin cartridges (typically 30–35 %) also contributed to the frequency of gloving in Australia and

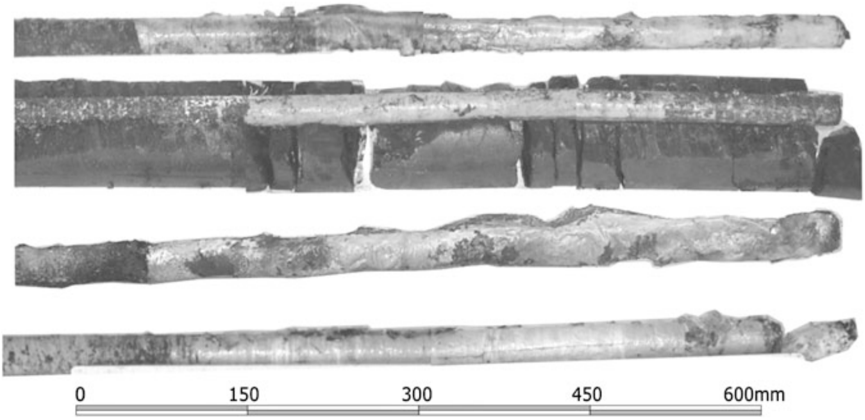


Fig. 6.50 Typical appearance of gloved and unmixed resin anchors (After Campbell et al. 2004)

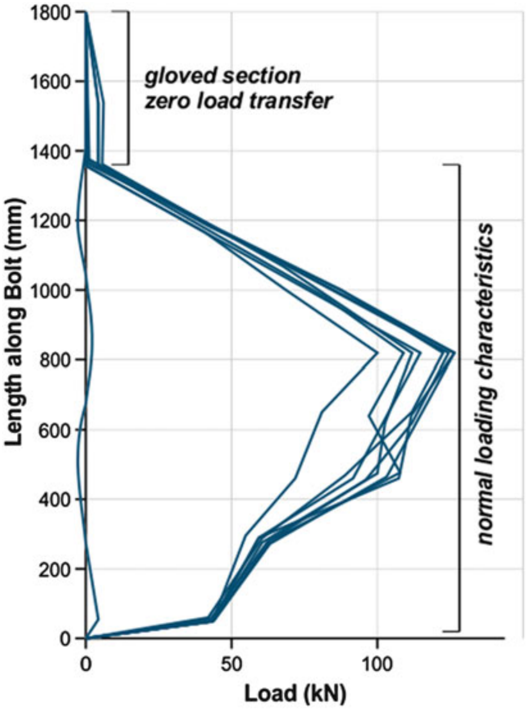


Fig. 6.51 Confirmation using an instrumented roof bolt of the lack of load transfer over a gloved portion of a bolt (After Campbell and Mould 2003)

New Zealand. Research by McTyer (2015) indicated that 12–15 % of installations were affected by gloving in the case of 24 mm nominal diameter bolts installed in 28 mm diameter holes using resin cartridges with a 1:15 ratio of catalyst-to-mastic. This decreased to 2 % when the bolts were installed with a 1:2 ratio mix. It was also found that the frequency of uncured resin in 28 mm and 30 mm diameter holes was 5 % and 26 %, respectively, when using a 1:15 ratio mix, as compared to only 0.2 % in both situations when using a 1:2 ratio mix.

ACARP (2014a) reported interim research findings that bolts installed in holes overdrilled by 50 mm resulted in higher load transfer capacity for the given installation. In particular, improvement in the load transfer capacity occurred near the back end of the installed bolt, where shredded plastic skin material accumulated inside the 50 mm of overdrilled hole above the bolt. Possible means for managing the risks associated with gloving include provisions in the Ground Control Management Plan for installing additional tendons based on a probabilistic assessment of the likelihood of gloving and/or longer tendons so that the gloved sections are not located at critical horizons.

#### 6.4.4.3 Corrosion

Chemical corrosion of the surface of a tendon due to agents such as oxidation and acid water reduces tendon capacity and life expectancy. Tubular tendons are at greater risk because of their increased surface areas. A range of treatments is available to minimise exposure to corrosion, such as galvanising, epoxy coating and plastic sleeving. In general, however, these treatments are costly, vulnerable to damage in a mining environment, and increase installation time. Hence, they are not compatible with large-scale tendon usage in underground coal mining, and full encapsulation of the outer and inner surfaces of tendons remains the primary defence against corrosion in this environment.

Pull tests do not provide adequate reassurance of tendon integrity, especially when tendons have been fully encapsulated. Ruppel and Wittenberg (2001) and Hartman et al. (2010) report on progress in developing non-destructive vibration and sonic based devices intended to address this problem. In the absence of such technologies, reliance has to be placed on spot checks involving recovery of installed bolts by overcoring and on observations at deteriorating or failed sites.

In the early 1990s, broken bolt ends started to be detected on a regular basis at a number of pillar extraction and longwall mines in Australia. Detection was confined to the unencapsulated portion of the bolts and it was not known how many bolts were broken further up the hole. Failure could occur within months of installation at loads well below design failure load and took on the appearance of brittle fracture with little necking. Research by Crosky et al. (2002) attributed the problem to **Stress Corrosion Cracking** (SCC) which they described as slow, progressive crack growth under the application of a sustained load in a mildly corrosive environment. Eventually the crack reaches a critical depth at which the remaining tendon section cannot support the load and rapid failure occurs at well below the ultimate tensile strength of the material.

Crosky et al. (2002) reported that fracture depth in bolts varied from <1 mm to over 40 % of the cross-sectional area of the bolt. The fracture pattern has the appearance of a dark thumbnail at the site of the initial crack that develops into a starburst pattern (Fig. 6.52a).

In the mining industry, SCC is usually taken to include Hydrogen Stress Cracking (HSC), or hydrogen embrittlement. In HSC, hydrogen enters the bolt from the local environment and accumulates until a critical concentration is reached, whereupon a crack forms and propagates to the limit of the hydrogen rich zone. Hydrogen ingress into the fresh crack ultimately leads to sudden failure of the bolt. The hydrogen source in a mining environment is often attributed to water containing sulphate reducing bacteria that generate $H_2S$.

For SCC to occur, there must exist simultaneously a susceptible material, a corrosive environment and an applied or residual stress (Craig et al. 2010). The SCC cracking detected in the early 1990s initiated at natural stress raisers, particularly at rebar patterns and threads, and usually on the tension side of a bend in the tendon, as evident in Fig. 6.52b. Research indicated that the problem was almost entirely confined to high tensile bolts and that bolts with a low fracture toughness were particularly susceptible to stress corrosion cracking. The majority of

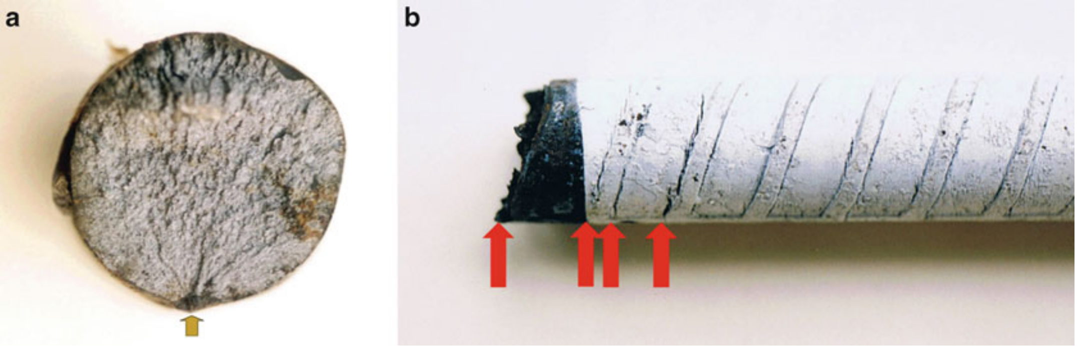


**Fig. 6.52** Appearance of stress corrosion cracking (SCC). (**a**) Fracture pattern, (**b**) Fracture locations (After Craig et al. 2010)

failed bolts were found to have very low Charpy impact values of 4–7 J (Crosky et al. 2002). Crosky et al. (2004) reported that when the Charpy value of new bolts was increased to 16, no failures were recorded for more than 2 years. This value has now been increased to 27 J, resulting in an enormous reduction in incidence of failure (Crosky et al. 2012).

While the problem appears to have been confined to high tensile bolts in Australia, the potential for other forms of SCC should not be overlooked. During the same period, HSC was associated with a fatal accident involving the sudden failure of mild steel bolts anchoring a conveyor loading point to the floor in a wet environment in another jurisdiction. Crosky et al. (2012) also associated incidents of SCC in Britain with the presence of hydrogen sulphide, noting that the failures did not generally initiate from corrosion cracks but rather from corrosion pits. Their research also suggested that there is a higher incidence of SCC at the site of clay bands within the bolting horizon, with this being related to clay mineralogy and potential electrochemical environments. More recent research findings are presented in Elias et al. (2013).

#### 6.4.4.4 Post-grouting Cables

Cables can be post grouted by either bottom-up grouting, with air being bled from the back of the hole through a breather tube, or by top-down grouting, with a tube being used to place grout at the back of the hole. Bottom up grouting provides assurance that the hole is fully grouted when grout returns down the breather tube. However, if the ground is fractured or parting planes are present, grout may be lost into these defects in preference to flowing to the back of the hole and fully encapsulating the cable. This can lead to pressurisation of the roof and increased roof convergence. Top-down grouting largely overcomes the problem of loss of grout into defects but requires the use of a thixotropic grout and does not permit verification of full encapsulation. In highly fractured and delaminated strata, it can be judicious to inject fractured ground with a strata binder or void filler prior to drilling cable bolt holes in order to limit grout loss during cable bolt installation.

#### 6.4.4.5 Other Performance and Safety Precautions

A range of other performance and safety related considerations apply to the installation and use of tendons. In particular:

- An under-drilled hole results in the tendon protruding an excessive distance into the working area. The risk associated with personnel and equipment coming into contact with these protrusions can be very high, especially at lower mining heights.
- The amount of pretensioning induced in a tendon by applying torque to the nut is a function of friction between the nut, the tread and the washer. If these components are damaged or dirty, the tendon may not be pretensioned effectively.
- The process of tightening a tendon can generate considerable heat which, if the drill hole is discharging flammable gas or if methane layering is present, can result in frictional ignition of the gas.
- Cables that are coiled to facilitate transport can present a whiplash risk when being uncoiled in a confined space underground.
- When high tensile bolts fail in bending or shear they can behave as projectiles. There is potential for this to occur when these types of bolts are used in applications such as supporting the roof in front of advancing longwall supports, anchoring conveyor equipment, and as anchor points for lifting devices.
- Resin cartridges are sensitive to heat and temperature and age with time. They need to be stored appropriately and not used beyond their expiry date. Underground workings often provide a good environment for storing resin cartridges.
- Resin loss in cracks during installation, overspinning, under-spinning, and gloving can all adversely impact on anchorage capacity and tendon performance.

- When testing anchorage products, it is important that the tests are conducted in the same horizons as those associated with full scale use.
- When utilising a two component anchorage product, the tendon should always be spun through the product, rather than pushed, to facilitate mixing and to minimise the risk of gloving.
- When tendons are installed at an angle to the rock surface or the rock surface undulates, plates should be used in conjunction with a hemispherical seat to minimise high contact stresses and bending of the tendon.
- In situations where wet drilling washes out strata, causes swelling of strata with resultant loss in borehole diameter, or produces a thin film of low friction material on the drill hole walls, consideration should be given to changing to a dry drilling process, with vacuum collection of drill cuttings.

## 6.5 Surface Restraint Systems

### 6.5.1 Scope

Surface restraint systems include cross supports, mesh screens, and membranes and liners. These types of systems are used to:

- prevent jointed and laminated strata from unravelling in between standing supports and tendons;
- seal strata that is prone to weathering;
- protect against brat (scat) and thin plies that scale (fall) from the roof;
- protect against falls of rib;
- carry deadweight load and redistribute it between primary support elements;
- restrain highly fractured and pulverised strata associated with geological disturbances; and
- stitch brows and open joints.

### 6.5.2 Cross Supports

The earliest forms of cross support were half round and full tree trunks, sometimes hewed to a square cross section, and referred to as **baulks**. These have now been replaced by W straps (or purloins), I beams, top hats, trusses and cable slings. Figure 6.53 shows examples of some of these cross support systems.

W straps typically range in thickness and width from 0.9 to 3 mm and 190 to 300 mm, respectively. While the W profile imparts some rigidity to make handling easier, this is of little benefit in terms of their support capacity. They can tolerate considerable convergence but are prone to tear under high load. The jagged and sharp surfaces that result introduces a new risk into the workplace.

The point load capacities of a range of other cross supports are presented in Table 6.6. Top hats are effectively thick W straps rolled to a profile that imparts a higher moment of inertia and, therefore, a higher resistance to bending. They were developed as a stiffer, stronger and, in some cases, lighter alternative to timber, steel rails and I beam cross supports. Top hats beams were designed originally to transfer deadweight load from above a roadway and into the floor via large timber legs set at each end of the beam. However, since the advent of cable bolting in the underground coal mining sector, top hats may also be suspended with cable bolts, with or without timber legs. Heavy sections with a high bending moment are often utilised to support deadweight load when the roof has delaminated to a height exceeding the length of tendons, in localised high stress zones, and when mining through major geological structures. They also find application in securing the lips (edges) of a roof fall to prevent its extension.

Trusses and slings find most application in highly laminated environments, such as that shown in Fig. 6.54, where beam building can be particularly difficult due to the very low shear strength of the bedding planes. Roof failure

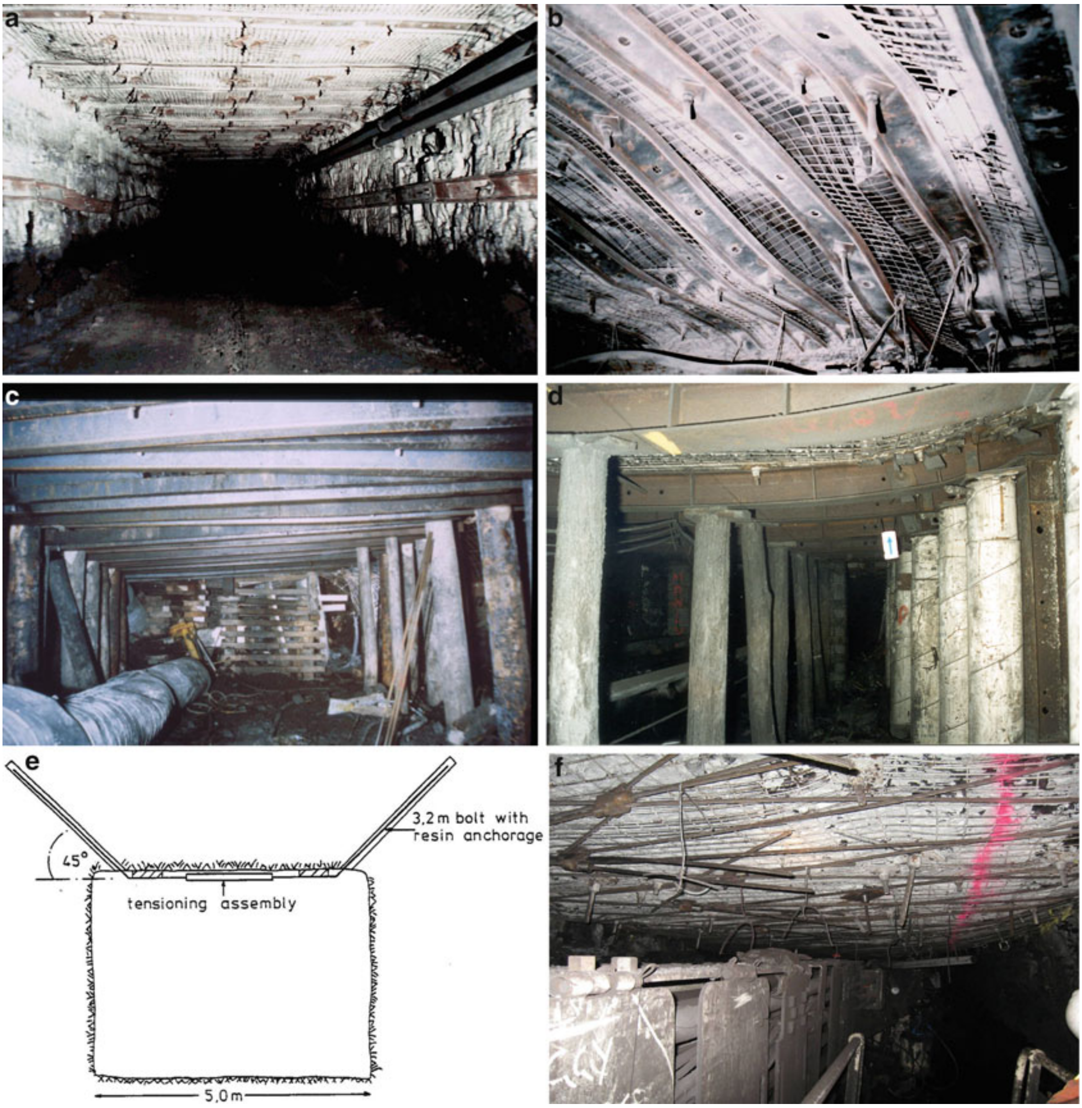
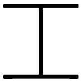
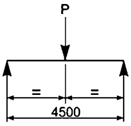
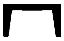
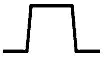
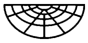


**Fig. 6.53** Examples of types of cross support. (**a**) W straps being used to restrain both roof and ribs, (**b**) W straps and mesh used to prevent the unravelling of roof strata between rock bolts, (**c**) Top hats set on heavy timber legs and also bolted to the roof, (**d**) Steel I beam cross supports with reinforced gussets carried on steel I beam legs supplemented with yielding pumpable legs, (**e**) Birmingham rod truss, (**f**) 10 m cable bolts with tails joined and tensioned to form slings

occurs on steep planes close to the excavation abutments. It can extend many metres into the roof due to the slow rate that excavation width reduces with caving height and the low bulking factor of laminated strata. The situation is aggravated by horizontal stress.

Experience has shown that roof trusses can be very effective in maintaining roof control in these circumstances. Figure 6.53e shows a typical roof truss arrangement that performed well in South African conditions (Galvin et al. 1982). Noteworthy features are the limited roadway width of 5 m; the inclination of the fully encapsulated rock bolts at 45° out over the solid abutments; and the trussing of the bolts by a rod assembly tensioned to form an integrated support system across the width of the excavation. Early concepts that a rod truss functioned by inducing

**Table 6.6** Typical load carrying capacities of various cross supports (Gale and Matthews 1993)

| Type & Size of support | Material specification | Section form | Weight (kg/m) | Yield load (kN) | Deflection (mm) | Method of load determination |
|---|---|---|---|---|---|---|
| BHP<br>200 UC<br>46.2 | AS 1204 G250 |  | 46.2 | 104 | 22 |  |
| BHP<br>200 UC<br>46.2 | AS 1204 G350 | | 46.2 | 144 | 30 | |
| BHP<br>150 UC<br>37.2 | AS 1204 G250 | | 37.2 | 63 | 27 | |
| BHP<br>150UC<br>37.2 | AS 1204 G350 | | 37.2 | 88 | 38 | |
| T'Makers<br>S.H.S<br>152 × 9.5 | AS 1204 G350 | | 40.2 | 59 | 39 | Note: |
| T'Makers<br>S.H.S<br>102 × 9.5 | AS 1204 G350 | | 32.6 | 26 | 58 | |
| BHP<br>47 kg Rail | AS 1085 | | 46.6 | 97 | 58 | 1. Beam simply supported |
| BHP<br>41 kg Rail | AS 1085 | | 40.7 | 70 | 51 | |
| BHP<br>Channel<br>152 × 76 | AS 1204 G250 |  | 17.9 | 5 | 42 | 2. Self weight ignored |
| BHP<br>Channel<br>152 × 76 | AS 1204 G350 | | 17.9 | 7 | 58 | |
| Aquila<br>Mine<br>Beam<br>230-8 | AS 1204 G250 |  | 56.5 | 118 | 21 | 3. Loaded to yield point |
| Aquila<br>Mine<br>Beam<br>185-6 | AS 1204 G350 | | 35.5 | 83 | 31 | |
| Aquila<br>Mine<br>Beam<br>150-6 | AS 1204 G350 | | 28.3 | 51 | 40 | |
| ½ Round<br>Timber<br>Ø300 | HARDWOOD |  | 27.0 | 71 | 250 | Physical test results as detailed in "Roof support in coal mines" by the NSW Coal Mines Safety Advisory Committee |
| ½ Round<br>Timber<br>Ø270 | HARDWOOD | | 22.0 | 62 | 280 | |
| ½ Round<br>Timber<br>Ø215 | HARDWOOD | | 14.0 | 27 | 250 | |

lateral compression in the roof have largely been superseded and it is now believed that the rod acts more as a sling (Naismith 1989; Tadolini et al. 1998).

Cable bolts can be made to perform a similar function by either connecting a pair of cables with a rod at their collars or leaving several metres of cable protruding from the drill holes and trussing and tensioning these tails (Fig. 6.53d). Cable trusses have the advantage that if the immediate roof loses its self supporting capacity, the sling component can tolerate a large amount of convergence while still preventing the roof from unravelling. Both rod and cable trusses may impart some confining stress to assist the rock mass to be self supporting higher up into the roof.

### 6.5.3 Screens

Screens comprise various forms of metal and synthetic mesh, synthetic nets, and woven mesh and mats. They find application in preventing strata from unravelling in between primary support elements (bolts, cables, straps), generating confinement to dilating strata, and restraining loose material (scats, slabs etc.) so that it does not present a fall of ground risk to personnel. Examples of the type of ground conditions that can benefit from support systems that incorporate screens are shown in Fig. 6.55.

Welded steel mesh has a significantly higher load carrying capacity and is more resistant to damage than synthetic mesh. However, it is relatively rigid and can be difficult to handle in confined spaces and to install on uneven rock surfaces. The grid size of welded wire mesh typically ranges from 50 to 150 mm, with 100 mm being common. Wire size ranges from 3 to 8 mm. Some mesh modules incorporate thicker wires that coincide with row spacing, with the tendons acting against these larger wires. Investigations by Robertson et al. (2003) have established that depending on wire diameter, welded steel mesh of 100 mm grid size has the capacity to support around 1 m of deadweight load if the mesh is pinned to the roof at 1.2 m centres. The research findings are summarised in Fig. 6.56.

Synthetic mesh is lighter but prone to tear and unzip if it comes into contact with equipment or sharp surfaces. It finds application mainly for rib support in situations where the coal is to be subsequently extracted. Welded steel mesh may still be utilised in these situations if ground conditions are particularly adverse.

Specialised synthetic screens and mats find application in protecting against goaf flushing in longwall recovery roadways. Photographs of this application are presented in Figs. 9.23 and 9.24. Synthetic screens based on fishing net and incorporating rope lacing have also proven successful as secondary support to protect against scats and roof falls in both coal and tabular metalliferous mines in South Africa. An example is shown in Fig. 6.57.

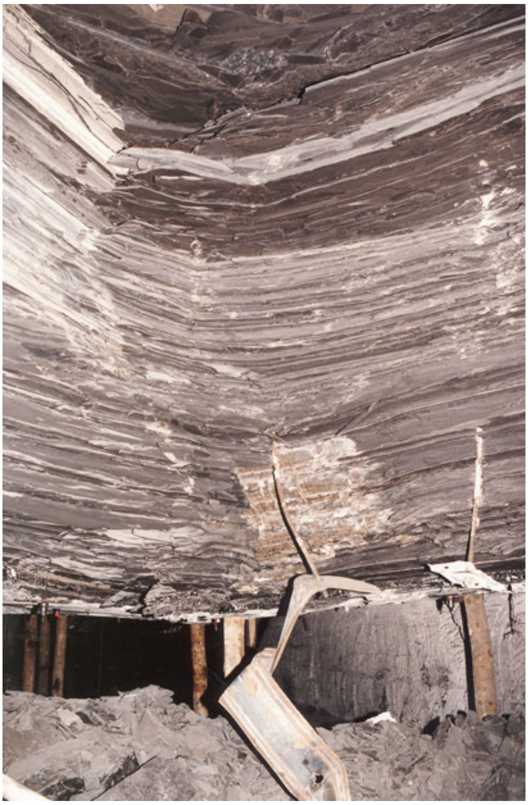


**Fig. 6.54** A roof fall site in highly laminated roof strata

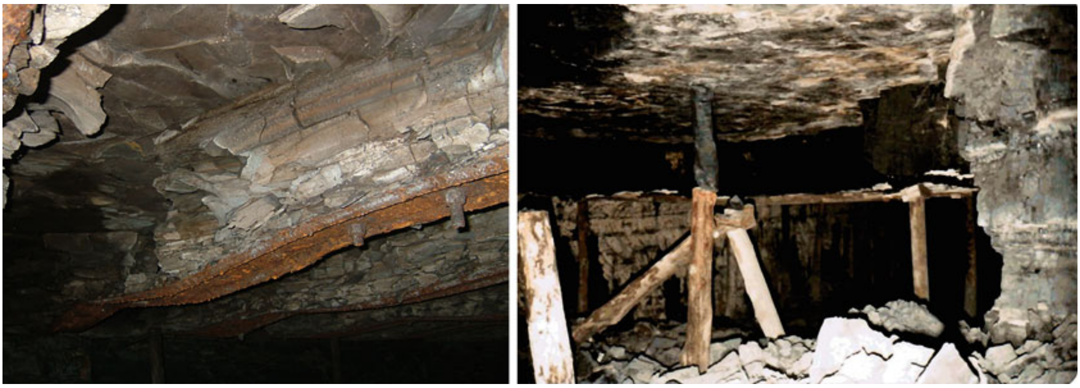
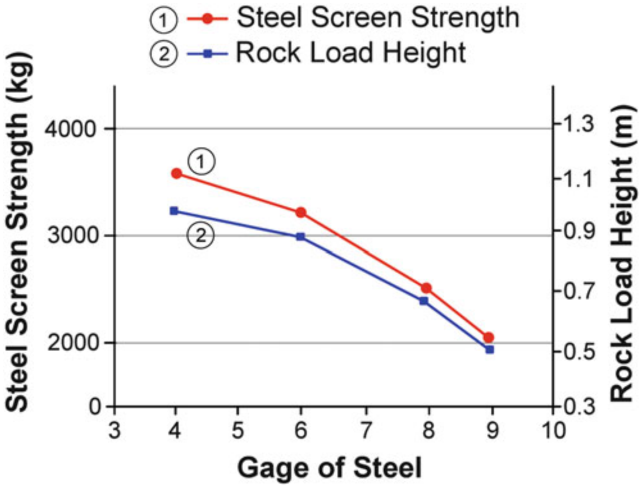


**Fig. 6.55** Examples of the types of roof conditions that can benefit from support systems that incorporate screens

**Fig. 6.56** Relationship between the gauge of 100 mm grid size welded mesh and deadweight load capacity when the mesh is pinned at 1.2 m centres (Adapted from Robertson et al. 2003)



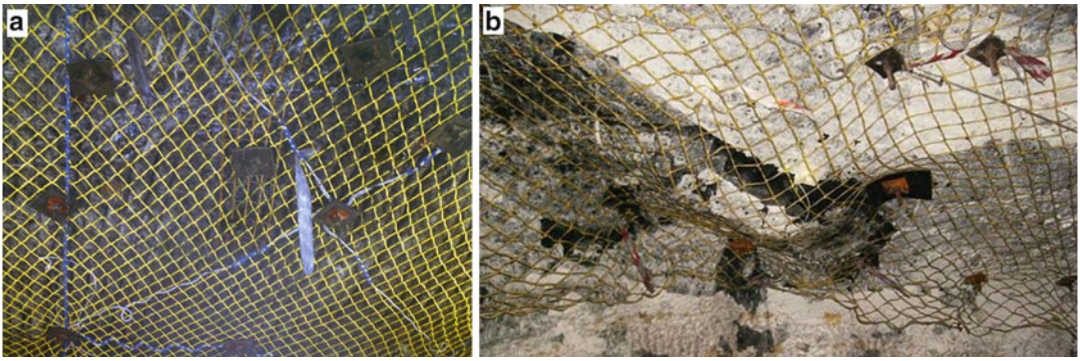


**Fig. 6.57** Purpose designed and manufactured synthetic fishing net incorporating cables for use as secondary support in old workings to protect against scat and falls of ground. (**a**) Net incorporating lacing installed as secondary support, (**b**) Small fall of ground retained by net

### 6.5.4 Membranes and Liners

Membranes and liners comprise shotcrete and thin spray-on liners (TSL). The applicability and performance of any type of membrane support, including shotcrete, in an underground coal mine is governed by:

- bond strength (adhesion);
- flammability;
- toxicity;
- sensitivity to temperature and humidity;
- set time;
- tensile strength;
- elongation;
- tear strength; and
- durability.

Wire mesh and fibre reinforcement can be used to increase the tensile strength, shear strength and ductility of shotcrete. Although shotcrete is used extensively and routinely at the working face in hard rock mining operations, it has found limited application in underground coal mining as a primary support measure, other than for supporting sites of major infrastructure. Basically, this is because coal mine working faces typically advance at a rate 30 times or more faster than in the case of hard rock mining; coal mining operations take place in a much more confined space; the strata is generally weaker and rock faces are more prone to yield; and sidewall support needs to be installed and effective within a few metres of the face as it advances and, therefore, within a few minutes where place changing (cut and flit) is not employed. These features currently make the routine application of shotcrete at the coal face impractical.

In an attempt to address these limitations and to eliminate manual handling issues, time delays and costs associated with the use of steel mesh, considerable research has been undertaken into the suitability of thin spray-on liners (TSL) or membranes for underground coal mining environments. Membranes may be cementitious or polymer based or a combination of both. They comprise one or two component mixes which are sprayed to a typical thickness of 2–6 mm. TSL are generally characterised by high tensile strength properties; high elongation capacity, although this may have to be traded off by a reduction in tensile strength; and an adversity to humid conditions and damp surfaces (Laurence et al. 2000; Gelson and Mahoney 2001). The successful application of some TSL is also sensitive to temperature. The long term durability of the support system is yet to be proven in underground coal mining environments.

Baafi et al. (2014) reported that researchers have been able to demonstrate polymer based liners can be developed with equal or better mechanical properties than steel mesh. Furthermore, adhesion of the polymeric material to the substrate provides an additional reinforcement and confinement mechanism not present with steel mesh. Under the same loading regime, steel mesh was found to deform substantially more than any of the polymer composites tested in the research program. However, the researchers noted that at that point in time, some of the polymers may have properties which could preclude their use in underground coal mines.

## 6.6 Spiling

**Spiling** involves inserting a series of closely spaced bars, or **spiles** at the working roof horizon to form a false roof, or **verandah**. Historically, spiling has found application for recovering fallen ground (discussed in Sect. 11.14) but it now also finds application for advancing a coal face through disturbed ground in virgin conditions. One or more rows of spiles up to 8 m in length and angled up at about 5° are grouted into the face, with spile spacing in each row typically ranging from 100 to 300 mm. Examples are illustrated in Figs. 6.58 and 6.59.

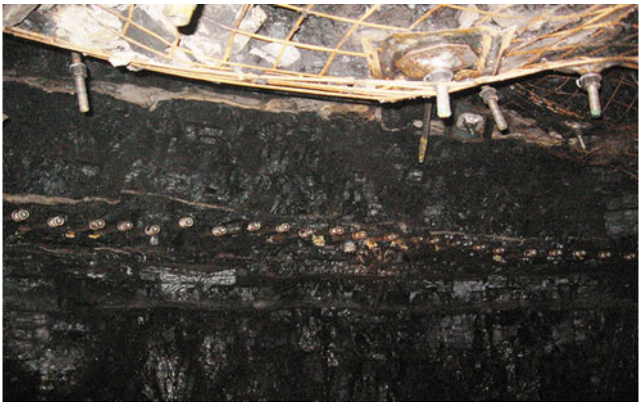


**Fig. 6.58** Spiles comprised of 2.5 m long self drilling bolts installed into the working face at the site of a roof cavity in order to create a protective canopy (verandah) to catch the lip as the face is advanced (After Corbett 2011)

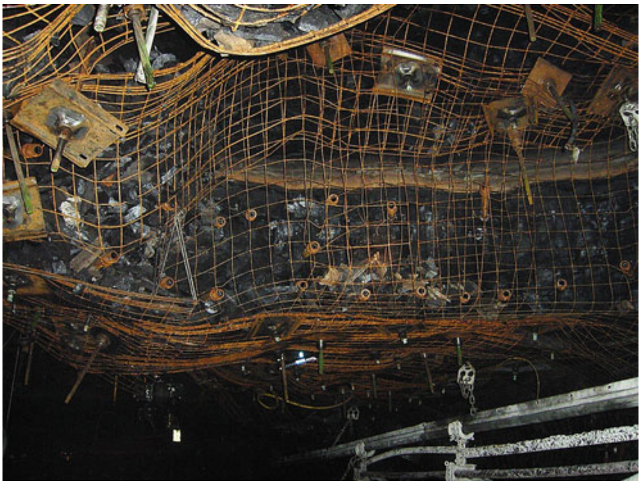


**Fig. 6.59** An example of the success of spiling to catch the lip at the site of a roof cavity (After Corbett 2011)

## 6.7 Strata Binders

The primary function of polyurethane resin strata binders is to consolidate and confine material that is already in a fractured state. They may also be used as a void filler to avoid loss of encapsulating medium when subsequently installing tendons. In most applications, the fractured material is usually in a stress relieved state, in which case the strata binder does not need to possess high compressive strength properties.

Strata binders generally comprise either microfine cement or polyurethane resin (PUR). Microfine cementitious based binders result in a stiffer consolidated rock mass but are slower to set and cure. As response time and not stiffness is more critical when consolidating broken strata, PUR finds the most application.

PUR was introduced into the USA coal industry in the late 1970s (Molinda 2008) and has been used extensively in Australia since 1986 when it was first utilised at Angus Place Colliery (Schaller and Russell 1986). Soon thereafter, the product was banned in Australia after an outbreak of fire at another colliery associated with using PUR as a bulk filler to treat a longwall face fall. PUR was also associated with a disastrous fire at Kinross Gold Mine in South Africa in 1986 that resulted in the loss of 177 lives due to the products of combustion of PUR. The ban was lifted in Australia in 1991 after a robust risk assessment process undertaken by Angus Place Colliery and the implementation of stringent controls. Approval is premised on limiting the amount of product injected into a hole, this value usually being 200 kg in Australia.

The product has the following attributes (Dalzell and Curth 1985):

- low viscosity, enabling injection into cracks;
- the increase in volume of the product after it has been placed can be varied from 1:1 to 1:12 at time of mixing;
- variable setting time from fast (seconds) to slow (hours);
- flexibility, ductility and excellent bonding properties;
- it can be formulated to work in wet ground (in fact, it is often used to seal water inflow pathways).

PUR has a high propensity to adhere to strata. Care is required in regard to the expansive nature of the product once it has been injected. The confining pressure generated by this action considerably increases the residual strength of fractured strata and contributes to the product's success. However, the same process has the potential to induce a fall of ground at the workplace. Schaller and Russell (1986) recorded a load increase of 15 kN (1.5 t) on a prop fitted with a stress cell at the injection site, with roof convergence varying from 1 to 7 mm, the upper value being measured during injection of the first hole. Controls for managing this risk during injection include:

- setting standing support in the workplace;
- limiting pump pressure;
- reducing the expansion ratio of the product;
- a trigger Action Response Plan premised on the remote monitoring of roof convergence.

## 6.8 Void Fillers

A feature of roof falls in underground coal mines is that they develop to the point where a stable state is once again achieved. This state arises because either the fall chokes itself off due to bulking, intercepts a competent bed, or domes out. In many instances, the fallen strata and the immediate surrounds of the fall are extensively fractured as a result of being subjected to high abutment stress. It is common for falls to occur after this abutment stress has been relieved, thereby removing confinement to the fractured material. During the process of removing material from a fall, operators are exposed to the risk of more material unravelling and falling from height into their workplace. This risk is also present if the sides and roof of the fall have to be re-supported.

Void fillers provide one means of managing these risks. Their two primary purposes are, firstly, to prevent failed material from unravelling, thereby enabling it to continue to provide confinement to the surrounding rock mass; and, secondly, to fill the void left by any fallen material which may have already been removed. The latter circumstance is often associated with cavities and falls on longwall faces. These functions do not require the material to possess high strength properties.

Most void fillers are based on either foaming cement or phenolic foam. Phenolic foam products are more expensive but have the advantages of a higher foaming capacity (up to 35-fold), quicker curing time and a thixotropic consistency. These translate to less materials usage and handling, reduced shuttering requirements, reduced operator exposure to the fall, and quicker placement rates. Both products enable fallen material to be stabilised to the extent that it can be re-mined. In the case of longwall mining, the time savings in recovering a face using a phenolic foam product can often

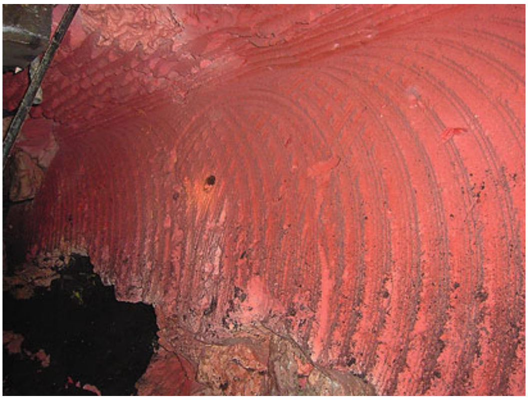


**Fig. 6.60** A phenolic foam product in the process of being mined on a longwall face where it was used as a void filler to successfully recover a roof fall

offset the additional cost of the material. Figure 6.60 shows an example of the effectiveness of a phenolic foam product in stabilising a longwall face fall.

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# Ground Support Design

# 7

**Abstract**

This chapter is concerned with assisting practitioners in evaluating and responding to their local ground conditions and site-specific behaviours and in drawing better informed conclusions as to the merits, limitations and reliability associated with support system designs. This approach also provides clarity to some contentious aspects of support system design. Support design considerations specific to pillar extraction and longwall mining are dealt with in Chaps. 8 and 9.

The five basic modes of roof failure in underground coal mining are identified. These may be interactive and some, in turn, may initiate a secondary mode of failure. A series of tables covering these failure modes is presented to provide guidance on identifying and responding to some of the more common types of roof behaviour in underground coal mines. The tables are supported with graphics of potential failure modes and photographs of these in underground coal mines.

Consideration is then given to a range of theoretical and operational aspects relating to roof support and reinforcement. These include the applicability and scope of classical beam theory; the role and timing of the installation of long centre tendons; aspects of some empirical based design methodologies; the effectiveness of pretensioning; the merits of numerical modelling; and stress relief.

Next, consideration is given to loss of rib control which in some countries such as Australia, accounts for the majority of serious and fatal falls of ground. Rib composition and behaviour are discussed along with a range of design considerations. The chapter concludes with a review of operational factors specific to coal ribs.

**Keywords**

Bedded strata • Blocky strata • Buckling • Cable bolt • Centre tendons • Centreline cracking • Classical beam theory • Cleat direction • Compressive failure • Cross measures • Euler buckling • Excavation span • Flexural failure • Guttering • Johnson’s formula • Mechanical

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advantage • Pretensioning • Reinforcement density indices • Reinforcement patterns • Rib control • Rib failure modes • Rib falls • Rib stability • Rock mass classification system • Roof bolt • Roof failure modes • Roof sling • Roof truss • Secant formula • Shear failure • Stress relief • Support patterns

## 7.1 Introduction

The design of ground support systems (which embraces both support and reinforcement elements) is concerned with maintaining free blocks of rock in place; preventing rock mass failure; controlling the extent of any rock mass failure; and supporting and confining failed material. Ground instability may be structurally driven, mechanically driven, or, as is most common in underground coal mining environments, a combination of both.

There are two primary means for controlling ground behaviour around an excavation, namely:

- reducing the mining-induced stresses through judicious mine planning, mine layout and mine scheduling; and
- mobilising the strength of the rock mass by conserving its inherent strength through the timely installation of support and reinforcement.

This chapter has a focus on ground support design for roadways in underground coal mines. Support design considerations specific to pillar extraction and longwall mining are dealt with in Chaps. 8 and 9, respectively. As with pillar support systems, excavation support systems are statically indeterminate. Therefore, the design of a support system is largely site-specific, being determined by the relative load-deformation characteristics, or relative stiffnesses, of the support system and the surrounding strata. For this reason, local experience in ground behaviour can be invaluable when designing ground support systems.

The design of ground support systems for roadways in underground coal mines has become controversial in some respects, with a range of approaches advocated. Some of these are diametrically opposed and some are defended dogmatically without due regard to principles of applied mechanics or to insight provided by numerical analysis. This creates a degree of confusion and uncertainty amongst mine site practitioners and, therefore, an element of risk. It can place mine managers, who have statutory responsibility for not putting the health and safety of employees at risk and management accountability for the economic performance of the business, in a difficult situation.

It is not the intention of this chapter to present or advocate specific support system designs. Rather, the chapter is concerned with providing guidance based on the physical and mechanical principles presented in earlier chapters. It is aimed at assisting practitioners in evaluating and responding to their local ground conditions and site-specific behaviours and in drawing better informed conclusions as to the merits, limitations and reliability associated with support system designs. This approach also provides clarity to some contentious aspects of support system design.

## 7.2 Roof Control

### 7.2.1 Failure Modes

The five basic modes of roof failure in underground coal mining are:

1. Gravity driven falls of unrestrained blocks of rock delineated by joints, bedding planes and mining-induced fractures.
2. Compressive (shear) failure of intact rock.

3. Flexural (tensile) failure due to excessive bending stress.
4. Abutment shear.
5. Buckling.

These failure modes may be interactive and some, in turn, may initiate a secondary mode of failure. For example, tensile cracking of a roof due to excessive flexural stress may lead to the formation of a linear arch, which can then fail in a number of different modes. Hence, a knowledge of failure modes and pathways is of considerable benefit in developing appropriate ground support strategies and designs to account for the changes in the function of support systems over the mining life cycle.

Root cause failure modes can be difficult to identify in the field and are often open to misinterpretation. Two examples are:

1. Bending stress failure in a clamped roof beam. The maximum tensile flexural stress in a clamped beam occurs on the upper surface of the beam at the panel abutments (see Sect. 2.8.3), where the failure planes prior to a fall of ground are out of sight. The appearance of the abutments of a roof fall in these circumstances can cause the fall to be incorrectly attributed to abutment shear failure.
2. Shear failure of stiff bands within the immediate roof strata. In an interbedded roof environment comprising soft and stiff bands, the stiffer bands both attract lateral stress and are prone to brittle failure. When a stiff band fails, the associated dilation can rapidly drive down the immediate roof, with this convergence being open to misinterpretation as buckling failure.

Tables 7.1, 7.2, 7.3, 7.4, and 7.5 provide guidance of a general nature on indentifying and responding to some of the more common types of roof behaviour in underground coal mines.

### 7.2.2 Generic Design Approaches

Ground support design approaches in underground coal mining cover the full spectrum of trial and error, experiential, empirical, analytical, semi-empirical and numerical. The reader is referred to Sect. 2.7 for a more detailed discussion of the generic merits and limitations of these approaches.

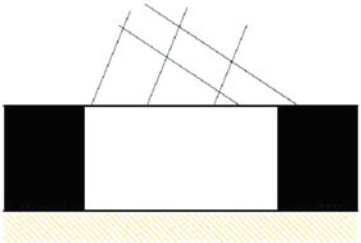
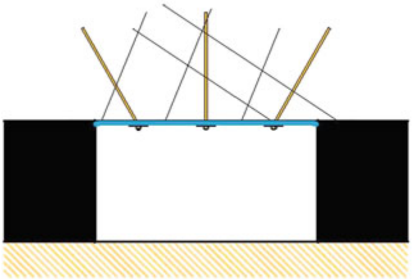
Trial and error continues to play a role in developing and testing new support technologies. From a risk management perspective, this approach requires that robust controls are in place to manage any risk associated with the error component.

Operational experience can be invaluable for developing and evaluating ground support design. Often, experienced operators have an ability to 'read the roof' and an intuition for selecting an appropriate support response. However, they may have little understanding of the mechanics of the behaviour they are responding to or why their support response is effective. Conversely, desk-top designers may have a good understanding of the applied mechanics principles but lack the operational experience to identify the ground behaviour conditions for which they need to design.

Empirical, or experimental, design approaches seek to establish correlations between support performance and raw field data or field data that has been processed in accordance with a recipe to produce a defined outcome such as, for example, a rock mass classification rating. Risk is intrinsic in this approach because of the potential for the collection of irrelevant, incomplete or incorrect data and for shortcomings in the engineering validity of correlations derived from that data.

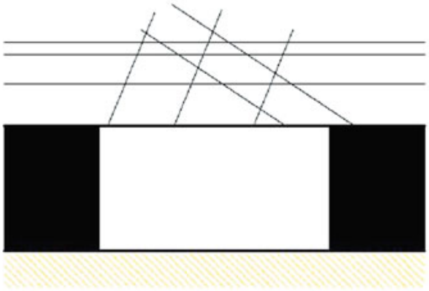
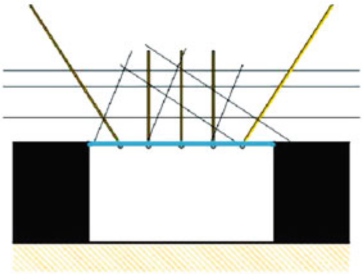
Analytical approaches are founded on applying applied mechanics principles to understanding ground behaviour and how this behaviour may be modified by mine design and the installation of support and reinforcement. This gives direction to the types of data that, ideally, need to be collected in order to place ground support design on an engineering foundation. The merging of these two processes constitutes what is referred to in this text as a 'semi-empirical approach' (but could equally be referred to as a 'semi-analytical' approach). Essentially, semi-empirical approaches to ground support system design are based on mechanistic models; select critical parameters; relatively simple and easy to

**Table 7.1** Models of discontinuity controlled roof failure

| Case 1(a) – Massive Blocky Roof | Generic Response |
|---|---|
| <br>Base Case – Two or more joint sets defining blocks which may drop out under gravity.<br> <br>Wedge failure associated with the intersection of two well defined joint sets. | <br>• Spot tendon support having regard to angle and direction that each tendon needs to be installed in order to retain blocks. May or may not include integrated cross support or surface restraint.<br>• Effectiveness is highly dependent on regular structural mapping and interpretation and on operator diligence in inspecting and interpretating joint patterns, directions and dips.<br>Alternatively<br>• Systematic tendon support, which may include integrated cross support or surface restraint.<br>• Some or all tendons may need to be long. |

(continued)

**Table 7.1** (continued)

| Case 1(b) – Bedded and Blocky Roof | Generic Response |
|---|---|
| <br>Base Case – Blocks defined by one or more joint sets plus bedding and that are prone to drop out under gravity.<br> <br>A fall of bedded and blocky roof | <br>• Systematic tendon support which may include integrated cross support or surface restraint.<br>• Some or all tendons may need to be long.<br>• Support system may need to be angled over the ribline in order to suspend immediate roof of excavation.<br>• Roof prone to fall with any relaxation of horizontal stress.<br>• Horizontal stress promotes drop out when joints dip at less than about 75º. |

(continued)

**Table 7.1** (continued)

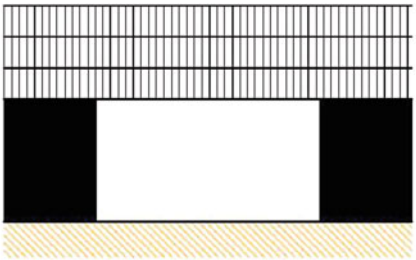
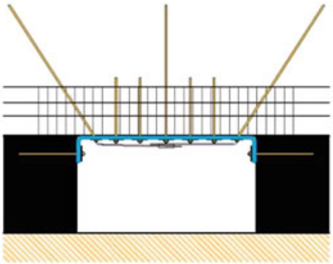
| Case 1(c) – Bedded and Jointed | Generic Response |
|---|---|
| <br>Base Case – Bedded strata that is intensely jointed and prone to unravel.<br> <br>A joint swarm | <br>• Systematic tendon support with surface restraint.<br>• Short tendons and surface restraint bind the beam together. Long tendons angled over the ribline and in the centre of the roadway transfer the weight of the beam to solid strata.<br>• Support system may include long centre tendons and trussed tendons angled over the ribline.<br>• Ribs may be adversely impacted by the same joint system and require full surface restraint, installed as close to the mining face as practical.<br>• Roof prone to fall with any relaxation of lateral stress.<br>• Horizontal stress promotes failure when joints dip at less than about 75º. |

**Table 7.2** Model of abutment shear roof failure

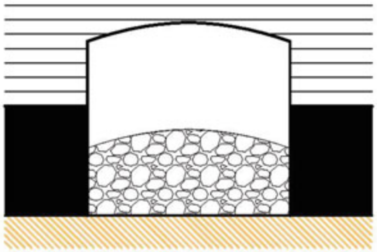
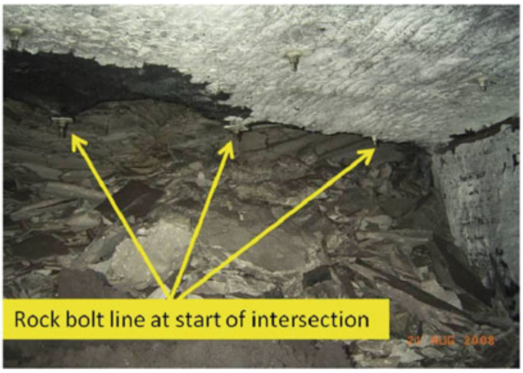
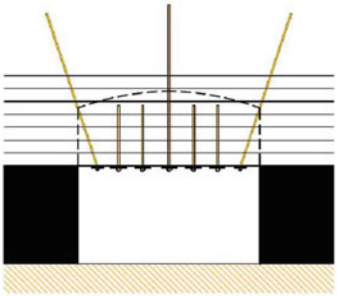
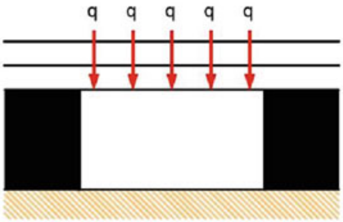
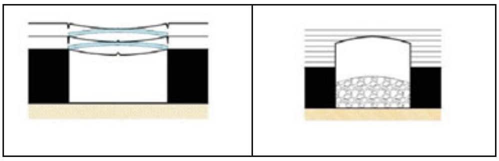
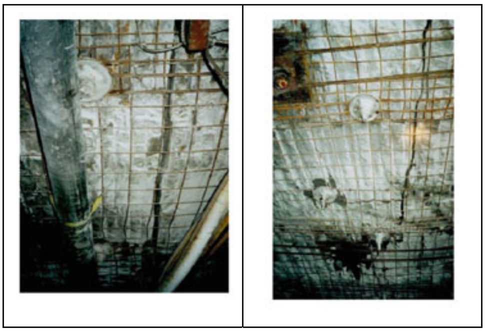
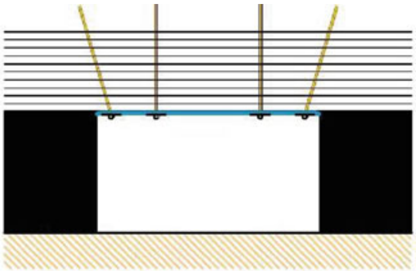
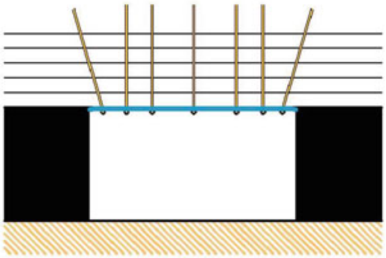
| Case 2(a) – Deadweight Driven Shear | Generic Response |
|---|---|
| <br>Base Case – Shear failure at abutments.<br> <br>Roof fall at intersection – note lack of signs of distress leading up to fall site<br> <br>Close up view of one lip of the above fall | <br>• Systematic tendon support. Short tendons build beam. Long tendons angled over the ribline and in the centre of the roadway transfer weight of beam to solid strata.<br>• May include cross support or surface restraint.<br>• Once a shear plane is mobilised, failure can occur rapidly.<br>• There may be few, if any, warning signs of impending failure, even when monitored with instrumentation.<br>• Prone to fall with any relaxation of lateral stress.<br>• Reinforcement strategy is focussed on tranferring deadweight load to abutments and providing adeqaute warning of the development of shear movement at the abutments. |

**Table 7.3** Models of flexure controlled roof behaviour

| Case 3(a) – Deflection Under Transverse Load | Generic Response |
|---|---|
|  <br>Base Case – Flexural failure due to bending under deadweight load . May or may not result in format ion of a linear arch (voussoir beam).<br><br><br>Centreline cracking down roadways.<br> <br>Linear arch principle in practice. | <br>• Support pattern designed to resist bedding plane shear when roof strata has a degree of capacity to span between tendons.<br><br>• Support pattern designed to resist bedding plane shear and to compensate for a lack of capacity to span between tendons.<br>• Provides local support to strata not confined within a linear arch. |

(continued)

**Table 7.3** (continued)

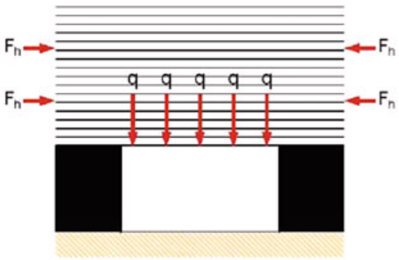
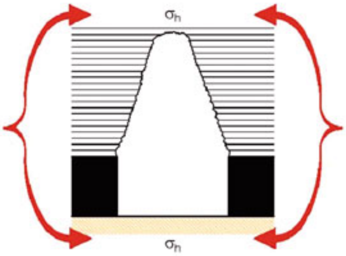
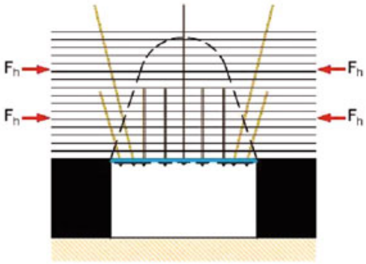
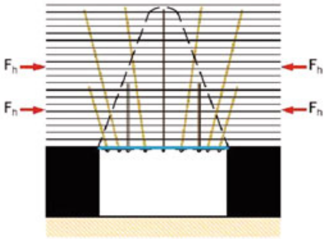
| Case 3(b) – Deflection Under Transverse and Longitudinal Load | Generic Response |
|---|---|
|  F<sub>h</sub><br>F<sub>h</sub><br>F<sub>h</sub><br>F<sub>h</sub><br>q q q q q<br>Base Case – Flexural failure due to bending and buckling under the combined effects of deadweight load and lateral stress.<br> <br> |  <br> <br>• Short and long tendons angled over the ribline with cross support, trussing and/or surface restraint.<br>• Long tendons in centre of roadway if they can be effectively anchored above zone of softening, otherwise in middle third of roadway.<br>• Short tendons, cross supports and surface restraint bind the immediate roof.<br>• Long central tendons increase resistance to buckling.<br>• Outer long tendons may be trussed.<br>• All long tendons transfer weight of beam to solid strata.<br>• Environment susceptible to floor heave. |

**Table 7.4** Model of compressive shear and dilation controlled roof behaviour

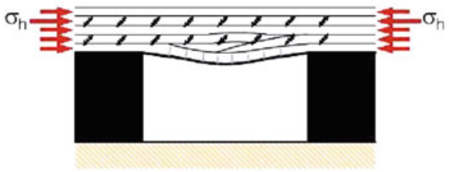
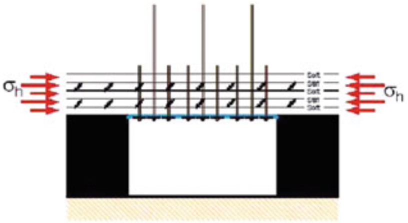
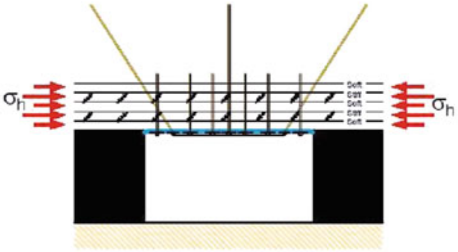
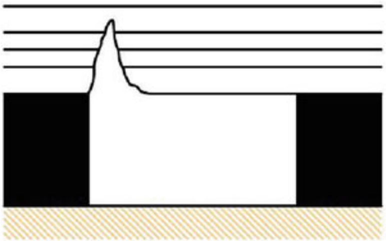
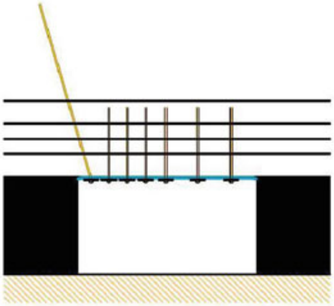
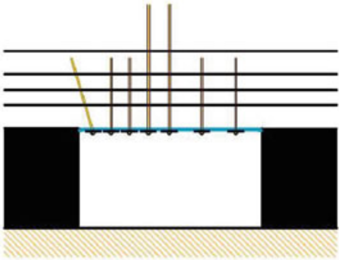
| Case 4(a) – Shearing of Stiff Band/s | Generic Response |
|---|---|
|  <br>Base Case – Compressive failure of stiff beds/s subjected to lateral stress, with dilation driving down lower strata.<br> <br>Alternating soft/stiff band environment<br> <br>Appearance of a roof fall at the mining face in a similar environment to above |  <br> <br>• Beam building and confinement of stiff bands using short and long tendons.<br>• Transfer of deadweight load of failed strata over excavation to solid strata over abutments using long tendons in the event that failure of stiff beds cannot be prevented.<br>• Often misinterpreted as a buckling failure. |

**Table 7.5** Model of guttering controlled roof behaviour

| Case 5(a) – Guttering – Shear Stress at Corners | Generic Response |
|---|---|
| <br>Base Case – Gutter due to high shear stresses in roadway corner, often associated with high lateral stress.<br> <br>Guttering | <br><br>• Tendon density biased towards gutter. Tendons may or may not be angled.<br>• Usually strata falls out before support can be installed and may continue to spall over time, therefore requiring secondary support.<br>• Long tendons may serve as both primary and secondary support.<br>• Gutter turns immediate roof into a cantilever which may then need to be suspended off long cables installed in centre third of roadway. |

use analytical models; field observations; and field performance evaluations.

Numerical modelling provides the scope to simultaneously evaluate a range of potential ground behaviour modes and failure pathways in two and three dimensions and to undertake parametric and sensitivity analysis to better quantify design risk. Serious reliability issues can still be associated with numerical modelling and, in general, it is not particularly user friendly from the perspective of mine operators.

Ground support design in the underground coal sector is dominated by semi-empirical and numerical approaches. Both are valuable but neither is without its flaws and limitations. Some of the more important of these are noted in the following sections as it is important that end-users have an awareness of them so that they can properly identify and assess residual risk associated with the use of the design approaches.

## 7.3 Theoretical Roof Support Design Aspects

### 7.3.1 Classical Beam Theory

Classical beam theory is a valuable aid for conceptualising and understanding how a roof beam may respond to the formation of an excavation and for evaluating ground behaviour during a site inspection. This is reflected in how it is utilised throughout this text to evaluate the significance of variables such as span, length, thickness, diameter and modulus on the mechanics and performance of structural elements. However, it is important not to lose sight of the numerous ideal assumptions on which classical beam theory is based (discussed in Sect. 2.8.2). These include that the beam is isotropic; homogenous; free from defects; linearly elastic; perfectly straight; of constant cross-section when in an unloaded state; initially stress free; of uniform rigidity; and only loaded normal to its faces. This set of assumptions is hardly ever likely to hold true in a coal mining environment.

The application of simple classical beam theory to the design of ground support and reinforcement systems has a number of additional limitations in a bedded environment. The theory is restricted in its capacity to evaluate the interactive behaviour of a series of beds of different flexural stiffnesses. Outcomes are particularly sensitive to end constraints, which can be variable in a mining environment and open to modification by the installation of reinforcement. For example, there is a fourfold difference between the deflection of a beam-column with clamped (fixed) ends and one with pinned (hinged) ends. In reality, neither end constraint is likely to apply in mining situations.

The most critical areas requiring effective ground support are the immediate face area and roadway intersections. The geometry and stress regimes associated with these mining environments impose additional constraints on utilising design procedures based on classical beam theory. In the vicinity of a coal face, the roof beam effectively comprises a plate with three clamped edges and an ill-defined cantilevered edge that is impacted by previously installed ground support and reinforcement. Furthermore, stress magnitudes and distributions are both highly variable, especially when roadways are not parallel to principal stress directions. Whereas classical beam theory is premised on imposing load on a structure, mining also introduces a significant component of unloading as a result of removing confinement. The situation can be just as complex at intersections, especially during their formation, with the end constraints for roof strata spanning intersections falling outside the scope of classical beam theory.

Hence, there are significant limitations associated with basing support and reinforcement design procedures on classical beam theory. In applying elements of this theory to design procedures, some confusion has developed around whether roof beams fail by bending or by buckling. This arises from at least one roof beam design procedure that attributes failure exclusively to buckling. Others argue that failure is partially and, in some instances, solely due to bending. Section 2.8 and Fig. 2.50 provide a basis for clarifying this confusion. In mechanics, the term ‘bending’ is associated with the deflection of a beam or column under the effect of transverse load, while the term ‘buckling’ is associated with the deflection of a beam or column under the effect of longitudinal load. In mining, the immediate roof beam can be subjected to either or both of these loading sources, which then generate bending moments that cause the structure to deflect.

As illustrated in Fig. 2.50, in the case of a transversely loaded beam, the bending moment is a function of distance, x, along the beam, being at right angles to the direction of deflection. On the other hand, the bending moment for a column is a function of the deflection, δ, and, therefore of distance in the z direction. In terms of the impact on the flexural loading in the beam, the source of the deflection is irrelevant. The outcome is still the same in that it results in an increase in fibre stress, given by Eq. 2.55. However, the source of loading is important when considering whether a beam or column is susceptible to elastic instability, or Euler buckling, and hence, to a step increase in fibre stress (see Sect. 2.8.4).

The extent to which immediate roof strata may be impacted by bending or buckling can vary with factors such as roadway width; the direction of the roadway relative to the strike of geological structures; and the direction and magnitude of principal stresses. Hence, the source of roof deflection needs to be assessed on a site specific basis and design procedures chosen accordingly.

When a beam or column is subjected to deflection, flexural failure is governed by the magnitude of the extreme fibre stress. In many structural engineering situations, the compressive yield strength, $\sigma_y$, of the material comprising a beam or column is the determining factor. However, in ground engineering, consideration also has to be given to the extreme tensile fibre stress generated in a rock beam or column, since the tensile flexural strength of rock is typically only one-tenth to one-thirtieth of the compressive yield strength. This is one of the factors that needs to be taken into account when applying strength formulations for intermediate length columns, such as the secant formula (Eq. 2.54) and Johnson’s formula. If the secant formula is invoked, for example, failure needs to be tested against both of the following two loading states:

$$\frac{P}{A}\left[1+\frac{e.c}{r^2}\,sec\left(\frac{L_e}{2r}\sqrt{\frac{P}{EA}}\right)\right] > \sigma_y \qquad (7.1)$$

and

$$\frac{P}{A}\left[\frac{e.c}{r^2}\,sec\left(\frac{L_e}{2r}\sqrt{\frac{P}{EA}}\right)-1\right] > \frac{\sigma_y}{T_f} \qquad (7.2)$$

where

$T_f$ *is a tensile strength reduction factor typically in the range of 10–30, depending on rock type and structure,*

*and:*

$P$ = *axial load* (N)
$A$ = *area subjected to axial load* ($m^2$)
$E$ = *elastic modulus* ($N/m^2$)
$L_e$ = *effective length* (m) (Eq. *2.42)*
$r$ = *least radius of gyration* (m) (Eq. 2.43)
$e$ = *pre – existing eccentricity* (m) (Eq. 2.52)
$c$ = *distance from central axis about which bending occurs* (m) (Eq. 2.54)
$\sigma_y$ = *compressive yield strength* (N)

Some coal mine support design procedures have regard to Johnson’s formula, given by Eq. 7.3. The formula was derived empirically for steel beams and, therefore, is focussed on a column failing in compression due to a combination of axial compressive stress and flexural compressive stress. In the ground support design methodology described by Colwell and Frith (2012), the same behaviour criteria determined for steel beams are applied to rock, these being that the yield strength, $\sigma_y$, is 0.7 times uniaxial compressive strength and, as shown in Fig. 7.1, the transition away from pure Euler buckling failure occurs at 0.5 times this yield strength.

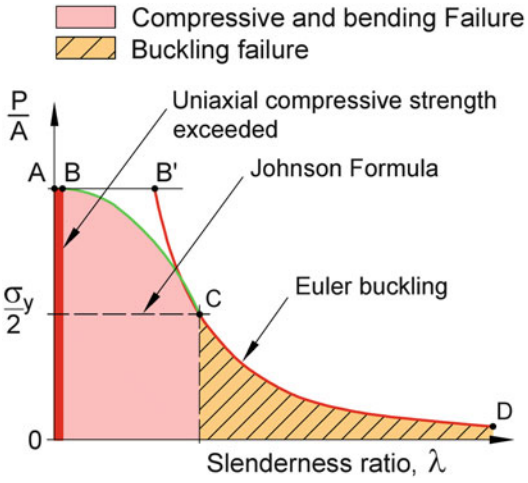


**Fig. 7.1** Failure mode transition based on Johnson's Formula and Euler's Formula

$$P_{cr} = \sigma_y A \left[ 1 - \left( \frac{\sigma_y}{4\pi^2 E} \right) \left( \frac{L_e}{r} \right)^2 \right] \tag{7.3}$$

where

$P_{cr} = \textit{critical load}$
$A = \textit{cross – sectional area of column}$
$E = \textit{elastic modulus of column}$
$\sigma_y = \textit{yield strength}$
$L_e = \textit{effective length of column}$
$r = \textit{the least radius of gyration} = \sqrt{I/A}$

If relying on this formulation, consideration has first to be given to how closely the behaviour of an intermediate length rock column might mirror that of a steel column. If the formulation is adopted then, as with the secant formula, criteria need to be developed for both compressive flexural failure and tensile flexural failure in rock.

### 7.3.2 Contribution of Long Central Tendons

Although long tendons are installed routinely in the central portion of roadways in underground coal mines, there is a range of views as to their function and mechanics of behaviour when installed in these locations. The applied mechanics principles developed in Sect. 2.8 provide a foundation for discussing some of these aspects and, in particular, the concept of 'mechanical advantage' as a design methodology. Some additional aspects of an operational nature relating to long central tendons are discussed in Sect. 7.5.3.

The contribution of a central tendon to ground support can be quantified by either applying simple beam theory (Sect. 2.8) or by equating the strain energy of bending to the work done by the applied load in displacing its point of application. The free body diagram for a simple beam theory approach is shown in Fig. 7.2, with the contribution of the tendon being found by solving for Eqs. 7.4, 7.5, and 7.6. A fuller description is presented by Jaeger and Cook (1979) but it is only a simple extension of the applied mechanics principles presented in Sect. 2.8, with the parameter 'k' representing the stiffness of the type of tendon employed (fully bonded, end-anchored, pretensioned, untensioned, etc).

$$EI\frac{d^2z}{dx^2} = P_{cr}(\delta - z) - \frac{1}{2}k\delta\left(\frac{S}{2} - x\right) \tag{7.4}$$

therefore

$$P_{cr} = \frac{4\alpha^2 EI}{S^2} \tag{7.5}$$

where α is the least root of

$$\tan(\alpha) = \alpha\left(1 - \frac{16EI\alpha^2}{kS^3}\right) \tag{7.6}$$

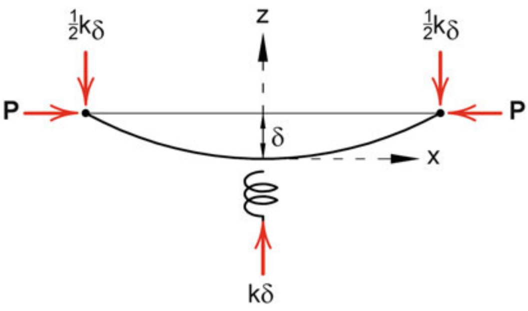


**Fig. 7.2** Free body diagram for an axially loaded beam with pinned supports subjected to elastic restraint at its midpoint (After Jaeger and Cook 1979)

In the strain energy method, the 'internal' energy, being the potential energy stored in the elastic deformation of the structure, is equated to the 'external' energy, being the work done on the system by external forces.

These applied mechanics approaches contrast with the concept of 'mechanical advantage' as developed and applied to rock beams in some roof and rib support design procedures developed for underground coal mines and described by Frith (2000) and Colwell (2004, 2012). The concept of mechanical advantage is well-established in engineering mechanics and used widely in engineering analysis and design. However, unless the analysis is based on a virtual work (or energy) approach, a number of limitations are associated with its application to rock beams.

This concept, as it is being applied to coal mine ground support design, assumes the beam to be weightless; to be pinned (hinged) at both ends; and to equate to a very thin, perfectly incompressible line that deflects in a perfect arc, with the end points of the line (or roof beam abutments) moving inwards with increased deflection (or curvature) of the line (Fig. 7.3). Hence, the length of the line, or roof beam, remains constant. The amount of inward deflection of the abutments under the effect of a lateral load, P, is designated '$U_{hi}$', with the corresponding mid-point line (beam) deflection, '$U_{vi}$', determined purely by a geometric relationship. It is proposed by proponents of the method that beam deflection, $U_{vi}$, generates a reaction force, *F*, with the equations of equilibrium for this system given by Eqs. 7.7, 7.8, and 7.9. The methodology is founded on the proposition that the ratio of vertical displacement to horizontal convergence ($U_{vi}/U_{hi}$) constitutes a mechanical advantage.

$$\frac{P}{F} = \frac{U_{vi}}{U_{hi}} \tag{7.7}$$

hence

$$PU_{hi} = FU_{vi} \tag{7.8}$$

and

$$F = \frac{PU_{hi}}{U_{vi}} \tag{7.9}$$

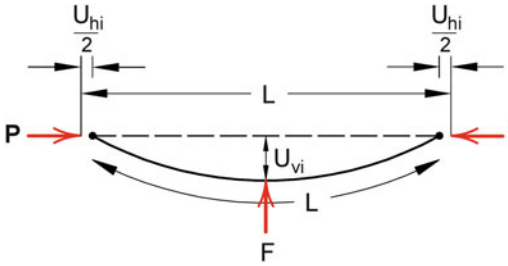


**Fig. 7.3** Geometric concept of mechanical advantage as described by Frith (2000) and Colwell (2004, 2012)

Figure 7.3 and Eqs. 7.7, 7.8, and 7.9 do not have regard to an appropriate free body diagram. For example, consideration is not given to the weight of the roof beam; the reaction forces, bending moments and shear forces generated at the beam ends; and to the forces generated between separate roof beams during bending, as illustrated by Brady and Brown (2006). These limitations become more serious as beam thickness increases. Similarly, so do limitations associated with the assumptions that the force generated in the tendon is distributed uniformly through the full thickness of the beam, and the redistributed horizontal stress is distributed uniformly both across the full width of the excavation and over the full thickness of the beam.

These factors aside, another way of expressing the fundamental relationships defined by Eqs. 7.7, 7.8, and 7.9 is to view the force, F, applied by the tendon at the mid-span as generating through mechanical advantage an additional axial thrust, −ΔP, given by:

$$-\Delta P = F\frac{U_{vi}}{U_{hi}} \tag{7.10}$$

Similarly, for a tendon located at a distance, x, from the ribside:

$$-\Delta P = F_x\frac{U_{xi}}{U_{hi}} \tag{7.11}$$

where

$F_x$ = *force applied by a tendon at position x*
$U_{xi}$ = *deflection at position x*

The geometric approximation between $U_{hi}$ and $U_{vi}$ in the published design procedures is not as mathematically precise as it could be. A more accurate and comprehensive approximation, derived in Appendix 8, is given by Eq. 7.12. Based on this formulation, 0.2 mm of lateral roof convergence would cause approximately 19 mm of vertical deflection at the midpoint of a 5 m long roof span and approximately 27 mm of vertical deflection at the midpoint of a 10 m long roof span. Thus, according to the concept, centre support systems would have mechanical advantages of 96 and 137, respectively.

$$U_{hi} = \frac{8(U_{vi})^2}{3L} \tag{7.12}$$

In order for the roof to deflect in an arc while not increasing in length, the end constraints of the beam have to be able to translate inwards. This behaviour is inconsistent with the assumption that the beam has pinned end constraints and with the assumed profile of the deflection curve associated with the mechanical advantage model. This curve closely matches a sine curve, defined by Eq. 7.13, which Timoshenko (1956) reports is a good approximation for the deflection of a beam that has simply supported ends (rather than fixed/clamped or pinned/hinged end constraints).

$$U_{xi} = U_{vi}Sin\left(\frac{\pi x}{l}\right) \tag{7.13}$$

where

$U_{xi}$ = *corresponding deflection at any point x along beam*

The mechanical advantage concept, as it is being applied to roof beams in the ground support design procedures described by Frith (2000) and Colwell (2004, 2012), is based on buckling of a column (or beam). It ignores the potential for bending to partially or totally account for deflection. Eq. 2.47 shows that the critical load required to induce buckling is inversely proportional to $(KL)^2$, where L is the length of the column and K is a factor equal to 1 for a column pinned at both ends and 0.5 for a column that is clamped at both ends. In a bedded or laminated coal mine roof, although the ribs may provide vertical restraint, slip between strata or beams may produce lateral translation near the ribs (Brady and Brown 2006). Furthermore, because of the partial clamping action of the strata immediately above and below the beam in question, the proposition that zero moment will always be transmitted at the pinned ends is questionable. This proposition becomes increasingly untenable as the thickness of the beam increases.

The design procedures associated with the concept of mechanical advantage attempt to account for the uncertainty surrounding the nature of the end constraints by adopting what is stated to be a conservative approach and basing calculations on the assumption of pinned end constraints. Care is required with this approach as it could be so conservative as to render the calculation of a factor of safety made on this basis to be almost meaningless.

Shanley (1967) presents another end condition which may be more applicable. As discussed in Sect. 2.8, this is one in which the ends of the beam-column may both rotate by a small angle, $\varphi$, and transmit a moment, $M$. In the mechanical analogue of this condition illustrated in Fig. 2.44, a coiled spring is used at the otherwise pinned end. The effective length coefficient, $K$, is a non-linear function of the product of the ratio, $M/\varphi$, and several other terms defining the beam geometry and its elastic properties. In most cases of practical interest, K values of between about 0.6 and 0.8 are likely to apply (Brown 2013).

Two serious deficiencies in how the concept of mechanical advantage is applied to roof beams in design procedures of the type previously noted are the failure to consider work done in shortening the beam by the additional axial forces generated by the tendon forces through mechanical advantage; and the failure to consider the bending deflection of the beam under its own weight. In the former case, strain energy is ignored. This is apparent from Eq. 7.8, since both $PU$ and $FV$ are expressions of energy. The only way that this equation can be satisfied is if no energy is consumed in the system, which is not mechanistically feasible. The situation may be analysed more accurately and conveniently using a virtual work approach, such as that described by Meriam (1975), rather than a force and moment–equilibrium approach. The conservation of energy equations should more correctly be written as:

$$\Delta PU_{hi} = FU_{vi} + W_1 \tag{7.14}$$

$$\Delta PU_{hi} = F_x U_{xi} + W_2 \tag{7.15}$$

*where*

*$W_1$ and $W_2$ are the values of elastic strain energy stored as a result of the application of the increments in axial force, $\Delta P$, arising from the application of the tendon forces, F and $F_x$, respectively.*

The exact values of $W_1$ and $W_2$ are not necessarily easy to calculate. However, simple stress-strain calculations show that the elastic strain energy stored as a result of this beam shortening can be large in comparison to the other terms involved in the energy balance equations.

With respect to the second deficiency, simple beam theory predicts that irrespective of lateral roof convergence, thin laminated beams will deflect considerably under their own weight. For example, beam theory predicts that a 5 m long, 0.05 m thick rock beam with a modulus of 4 GPa and clamped ends will deflect over 30 mm due to its self weight (see Appendix 3 for the formula). This is 50 % greater than the deflection attributed to lateral roof convergence in the earlier example in this section and upon which the concept of a mechanical advantage of 96 was premised. One consequence of bending is that it results in delamination of the roof strata, or roof softening, thus causing lateral stress that otherwise might drive lateral roof convergence and buckling to be redirected higher into the roof strata.

In summary, the concept of mechanical advantage as it is being applied in some ground support design procedures is incomplete from an applied mechanics perspective. Careful consideration needs to be given to the reliability of this concept.

### 7.3.3 UCS – E Correlations

With few exceptions, it is prohibitively expensive and impractical in underground coal mining to both obtain and test a large number of rock samples in the laboratory in order to determine 'representative' values of uniaxial compressive strength (UCS) and Young's modulus (E) required for analytical and numerical analysis. Hence, it has become common practice to estimate UCS using indirect methods, such as sonic velocity or point load testing, and to correlate E to UCS on the basis of a limited number of laboratory determinations of both values. Linear regression (as discussed in Sect. 2.7.5) is used extensively for this purpose.

This approach can result in considerable variability in predicted values of both UCS and E. The variability is to be expected since, as noted by Canbulat (2010, 2011), rock properties are not deterministic or unique values but are intrinsically variable and follow some form of probability distribution curve. The amount of scatter in the data is sometimes extreme, with the relationship between the two parameters having an unacceptably poor coefficient of determination, $r^2$. This can be associated with an attempt to apply a single linear UCS-E correlation to multiple rock types. These values can vary widely between rock types, even within sedimentary rocks, as shown by Hoek and Diederichs (2006) for example. Accordingly, it is preferable to develop specific UCS-E correlations for different rock types or groups of closely related rock types.

Risk assessment should have regard to the confidence that can be placed in the UCS-E correlations used in the design process and the sensitivity of outcomes to that correlation. A stochastic approach is preferable in quantitative risk assessment processes (see Sect. 2.7.5).

### 7.3.4 Rock Mass Classification Systems

The advantages and limitations of rock mass classification systems are discussed in Sect. 2.6.10. Regard needs to be had to a range of precautionary advice, including that of Hoek and Brown (1980), Brady and Brown (1985), Hoek et al. (1995), Hartman and Handley (2002), Pells (2008) and Suorineni (2014). These advices relate to the risks associated with applying rock mass classification systems without a proper understanding of their derivation, scope, limitations and the mechanics of the problem to which they are being applied. This includes their use in the design of ground support systems.

Seedsman (2003) noted that there are challenges with the use of rock mass classification systems for designing reinforcement systems because bedding, joints and rock strength are not mobilised simultaneously in any failure. Pells (2008) emphasised that rock mass classification systems do not constitute a structural approach for the proper design of rock bolts because they provide little or no idea of the loads that the reinforcement is supposed to carry and the shear and tensile displacements that the bolts are expected to encounter. These and other precautionary advices need to be borne in mind when selecting and assessing the level of risk associated with ground support design methodologies that have a reliance on a rock mass classification system.

### 7.3.5 Reinforcement Density Indices

A number of procedures relating to ground support design rely to some degree in their formulation on empirical data that has been processed on the basis of a so-called Reinforcement Density Index, RDI, of the general form given by Eq. 7.16. Examples relate to the calibration of ALPS – Analysis of Longwall Pillar Serviceability for application in Australia (Colwell 1998), the design and management of wide roadways (Thomas 2010), and ADFRS – Analysis and Design of Faceroad Roof Support (Colwell and Frith 2012).

$$\mathrm{RDI} = \frac{L \times N \times \mathrm{L}_{\mathrm{T\,peak}}}{S \times R \times k} \tag{7.16}$$

*where*

$L$ = *Installed tendon length* (m)
$N$ = *Number of tendons per row*
$\mathrm{L}_{\mathrm{T\,peak}}$ = *Ultimate tensile capacity of tendon* (kN)
$S$ = *Excavation span* (m)
$R$ = *Tendon row spacing* (m)
$k$ = *an adjustment factor for dimensions of parameters*

Careful consideration needs to be given to two potentially serious limitations associated with relying on this type of index for processing empirical data and developing design methodologies from it, as evident from the discussion of ground support mechanics in Chap. 6. These are:

1. There is an underlying presumption that the support patterns and densities observed in the field were near optimum and, therefore, provided a reasonably reliable basis for deriving a support procedure. As illustrated by Fig. 6.21, the number of tendons installed at a site is not necessarily a reliable indicator of the reinforcement required to maintain ground stability, with a similar level of displacement being associated with a simulated 4 bolt pattern and an 11 bolt pattern.
2. The index has little regard to the mechanics of behaviour. The potential implications that this could have for ground support design are readily apparent when it is considered, for example, that the RDI gives the same support rating to five 200 kN (20 t) capacity tendons; to two 500 kN (50 t) capacity cables each angled over opposite riblines; and to one 1 MN (100 t) capacity cable installed vertically towards one rib line. Field experience highlights the potential for this deficiency to impact design. For example, Payne (2008) reported that when weak roof was encountered at Crinum Mine, Australia, roof bolt density was doubled to 12 bolts per metre by halving the row spacing but, nevertheless, the roof continued to sag between the bolt closest to the rib and the second bolt in the row, usually on the stress impacted (notched) side of the road. However, reversion to a 1 m row spacing and installation of an extra bolt on each side of the roadway between the two outer bolts proved very effective in reducing roof movement and acting as a breaker line to roof falls.

The mechanics of how tendons function, the mechanics of the strata response, and the support types, densities and locations appropriate to these mechanics all need to be considered in the design process. This is particularly important in situations such as longwall installation roadways driven in multiple passes, where spans are large and the roof strata is subjected to a number of episodes of displacement, with the potential for the mechanics of the strata response to differ between episodes.

### 7.3.6 Numerical Modelling

Simple static limiting equilibrium analyses associated with many analytical and semi-empirical approaches to design essentially treat the system components as rigid bodies and use simplified models of system mechanics. More comprehensive computational approaches take into account the deformation and slip or yield of the support and reinforcing system elements and the rock mass. Both approaches find application in designing support and reinforcement systems for underground coal mining environments. Some highly empirical, analytical and semi-empirical support system design methodologies premised on classical beam theory have gained prominence since the early to mid 1990s. Nevertheless, the fact remains that, as argued by Brady and Brown (2006), a comprehensive analysis of rock reinforcement must be based on loads mobilised in reinforcement elements by their deformation and by relative displacement between host rock and components of the reinforcement. This is reflected to some degree in the approach to reinforcement design adopted by Pells (2008) and summarised in Fig. 6.29, albeit that this is still a relatively simple analytical approach.

However, it needs to be recognised that as well as being numerically intensive, the use of a comprehensive numerical modelling approach requires a wide range of input data detailing the boundary conditions, constitutive laws and material properties. Careful attention has to be paid to model formulation, input data, the solution procedure, and the interpretation and 'believability' of the outcomes. The nature of the numerical model and the input data need to be sufficiently transparent to enable the results to be evaluated critically by a third party. Subject to satisfying these criteria, numerical modelling provides a powerful tool for undertaking parametric and sensitivity analysis of input data and gaining insight into likely ground behaviours and appropriate support design methodologies.

## 7.4 Summary Conclusions

While techniques such as rock mass classification systems and regression analysis of masses of data can be useful in developing ground support design systems, they are not a substitute for an applied mechanics approach to ground support design. The optimal approach to the design of support and, in particular, reinforcement systems, is perhaps best summed up by Brady and Brown (2006). The authors state that although analytical solution may be of value in preliminary studies of a range of problems, most practical underground mining problems require the use of numerical methods for their complete solution. Frequently, support and reinforcement design is based on precedent practice or on field observations and experience gained in trial excavations or in the early stages of mining in a particular area. However, it is preferable that a more rigorous design process be used and that experiential or presumptive designs be supported by some form of analysis. Depending on the application, design calculations may be of a simple limiting equilibrium type or, ideally, be based on comprehensive computational approaches involving rock-support interaction calculations and taking account of the deformation and strength properties of the support and reinforcement system and the complete stress strain response of the rock mass (Brady and Brown 2006).

## 7.5 Operational Roof Support Design Aspects

### 7.5.1 Roadway Span

In bedded immediate roof strata, ground control tends to become more difficult as the density of the bedding planes increases or the strength of the individual beds decreases. There are two basic strategies for endeavouring to achieve a satisfactory degree of stability in this type of strata, namely:

1. limiting induced stresses in the roof though mine design, mine layout and mining sequence; and
2. increasing the apparent strength of the immediate roof zone by installing reinforcement and support.

One of the most effective means of improving roof stability, and one that applies to all mining environments, is to reduce the effective span of the excavation. This is illustrated by reference to classical beam theory (Sect. 2.8), with the formulations for beams, beam-columns and plates subjected to either or both transverse and axial load all reducing to the forms given by Eqs. 7.17, 7.18, 7.19, and 7.20.

$$Maximum\,Deflection\,(sag) = \delta_{max} = \frac{K_1 L^4}{E t^2} \tag{7.17}$$

$$Maximum\,Flexural\,(Bending)\,Stress = \sigma_{max} = \frac{K_2 L^2}{t} \tag{7.18}$$

$$Maximum\,Shear\,Stress\,(at\,abutments) = \tau_{max} = \mathrm{K}_3 L \tag{7.19}$$

$$Critical\,Euler\,Load = P_{crit} = \frac{K_4 t^3}{E L^2} \tag{7.20}$$

*where*

*E = elastic modulus of beam, column or plate.*
*L = length, or span, of a beam, column or plate.*
*t = thickness of beam, column or plate.*
*$K_1$, $K_2$, $K_3$ and $K_4$ are proportionality factors related to various loading configurations and end constraints.*

Table 7.6 shows the impact of excavation span on deflection, flexural stress, shear stress and critical buckling load in a roof beam as determined from Eqs. 7.17, 7.18, 7.19, and 7.20. Increasing the width of a 5.5 m wide roadway by only 0.5 m is sufficient to result in a 42 % increase in roof sag, a 19 % increase in bending stress and a 23 % reduction in the load required to induce buckling. Such an increase is common in driving roadways, especially when using a two pass continuous miner or when ribs are weak.

**Table 7.6** Assessment of the impact of excavation span and bedding thickness on deflection, flexural stress, shear stress and critical buckling load in a beam, beam-column and plate

| Situation | Dimension (m) | New dimension | Deflection | Flexural (bending) stress | Shear stress | Critical Euler buckling load |
|---|---|---|---|---|---|---|
| Base case width | B = 5.5 | – | 1 $\delta$ | 1 $\sigma_F$ | 1 $\tau$ | 1 $P_{cr}$ |
| New width | 2.75 | 0.5 B | 0.06 $\delta$ | 0.25 $\sigma_F$ | 0.5 $\tau$ | 4 $P_{cr}$ |
| New width | 6.0 | 1.09 B | 1.42 $\delta$ | 1.19 $\sigma_F$ | 1.09 $\tau$ | 0.77 $P_{cr}$ |
| New width | 7.8 | 1.42 B | 4.05 $\delta$ | 2.01 $\sigma_F$ | 1.42 $\tau$ | 0.50 $P_{cr}$ |
| New width | 11.3 | 2.06 B | 17.82 $\delta$ | 4.22 $\sigma_F$ | 2.06 $\tau$ | 0.24 $P_{cr}$ |
| Base case thickness | t = 0.3 | – | 1 $\delta'$ | 1 $\sigma_F'$ | 1 $\tau'$ | 1 $P'_{cr}$ |
| New thickness | 0.24 | 0.80 t | 1.56 $\delta'$ | 1.25 $\sigma_F'$ | 1 $\tau'$ | 0.51 $P'_{cr}$ |

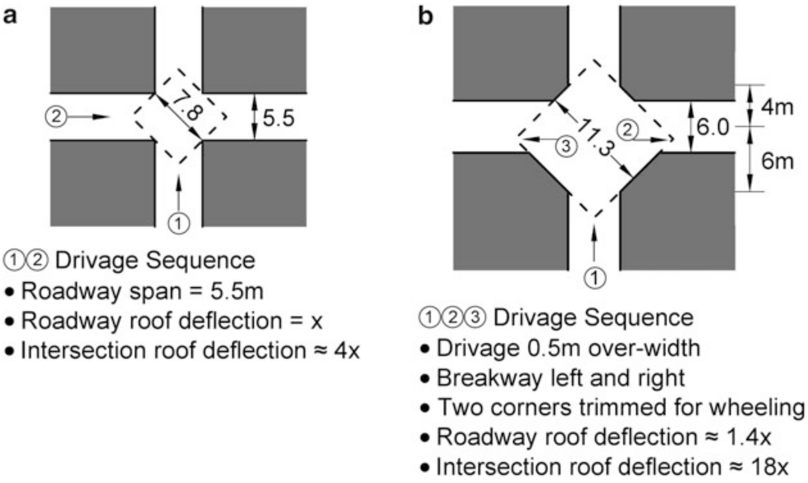

a
②
7.8
5.5
①
①② Drivage Sequence
• Roadway span = 5.5m
• Roadway roof deflection = x
• Intersection roof deflection ≈ 4x
b
4m
6.0
6m
11.3
②
③
①
①②③ Drivage Sequence
• Drivage 0.5m over-width
• Breakway left and right
• Two corners trimmed for wheeling
• Roadway roof deflection ≈ 1.4x
• Intersection roof deflection ≈ 18x

**Fig. 7.4** Effect of mining parameters on intersection span.

However, these impacts are relatively minor in comparison to those that occur at intersections. In theory, a 5.5 m wide roadway results in a 7.8 m span at a four way intersection as shown in Fig. 7.4a. However, in practice this is not achievable because the length of a continuous miner is typically two roadway widths and, therefore, cut-throughs cannot be driven off headings at 90°. Rather, the continuous miner has to cut its way into the cut-through in a fanning action, resulting in an increase in the span of the intersection (Fig. 7.4b). Other factors such as overwidth drivage; correction of off-centre drivage; rib spall; and trimming corners to facilitate wheeling also contribute to an increase in the span of a four-way intersection.

The effect of these operational factors is illustrated by reference to Fig. 7.4b, which shows an intersection with an 11.3 m span associated with a typical mining sequence. Disregarding the increased displacement arising from the difference in the end conditions of strata spanning a roadway and strata spanning an intersection, the increase in span still results in almost an 18-fold increase in roof sag, over a fourfold increase in bending stress, more than a doubling in shear stress, and over a 75 % reduction in the critical load required to induce buckling. This can have particularly serious implications for mining under weak roof strata or in high lateral stress environments. Hence, intersections present an elevated risk of instability, as borne out in practice. For example, Spearing et al. (2011) report that approximately 71 % of all roof falls in USA underground coal mines occur in intersections, even though intersections account for only 20–25 % of the roof area exposed by roadway development.

It follows that the manner and sequence of forming intersections, conformance between design and 'as-mined' dimensions, and rib control can critically impact on roof stability. The manner and sequence in which intersections are formed is influenced by a number of factors that include:

- Regulatory restrictions on roadway width.
- Ventilation systems. Variables include one sided or flanking panel return airways, forcing or exhaust ventilation, brattice or auxiliary fans, onboard scrubbers, air movers and blowers.
- Roadway development system. Options include cut and bolt or place changing.
- Conveyor belt heading position within the panel.
- Apparent seam dip. This impacts on whether intersections are broken away downhill, uphill or across a grade.
- Pillar shape. This determines the angle of the cut-throughs relative to headings.
- Pillar size. This determines flit distances and whether pillar centre distances are a multiple of cut-out distance.
- Physical size and manoeuvrability of equipment.

- Continuous miner remote control operator visibility and safe work location. Refer, for example, to MDG-5002C (2011).
- Wheeling routes and distances.

These types of factors determine the feasibility of the most preferable roof control measure of forming only one breakaway (from a flank heading) per cut-through, to forming only one breakaway at an intersection, through to the least preferable roof control measure of forming breakaways left and right at multiple intersections. They are particularly important considerations when selecting the type of roadway development system and the size and shape of coal pillars. Ground control management systems should make provision for the control of intersection span and the installation of additional support in over-width intersections.

### 7.5.2 Timing of Installation

From a mechanistic perspective, ground support is most effective in preventing and controlling the extent of rock mass dilation in bedded strata if it is installed as close as practical to the coal face and as soon as possible after mining. At that point in the mining cycle, the roof beam effectively constitutes a clamped plate supported on three sides. As such, it is subjected to less deflection and, therefore, less shear displacement between beds, than if it is allowed to develop into an unreinforced beam-plate as mining advances. Hence, the benefits of installing reinforcement will start to materialise at lower levels of roof deflection and time-dependent deterioration. While rock mass failure may still occur, the early installation of reinforcement aids in controlling the extent of this failure and in mobilising the inherent strength of the rock mass.

However, situations can arise where shear displacement is so large that it is not practical to install enough reinforcement to resist this movement at the face and prevent rock mass failure. This is often the case when mining in high lateral stress environments and at depth. If the reinforcement is installed too early or is too stiff, it will be rendered ineffective by shear displacement and deflection unless it has a good capacity to yield. While a range of techniques exist to incorporate yielding capacity into ground support elements, options are limited when it comes to tolerating the guillotining effect of shear displacement on tendons. One of these options is the installation of non-encapsulated tendons in larger diameter drill holes. In these circumstances, a balance may have to be found between installing appropriate and adequate ‘temporary’ support at the face while permitting the roof to relax sufficiently, but not excessively, before installing more permanent reinforcement.

The concept of permitting the immediate roof to fall to some limited height where it self-stabilises has been advocated by some, particularly when taking deep cuts, or plunges, in place changing operations. While this appears to have some merit, a number of practical aspects can give rise to elevated risk if the roof does not fall consistently to a well defined parting plane. The more obvious are exposure of operators to falls of residual roof and brat while installing support, and operational difficulties in installing support against an irregular roof profile. The less obvious risk, but arguably more important, is that this approach can result in operators becoming conditioned to risk. That is, the sub-optimum becomes the norm and the benchmark for poor decision making by operators in other situations (for example, when subsequently extracting pillars or recovering a longwall face).

### 7.5.3 Role and Timing of Centre Tendons

In Sect. 2.8, it was noted that the longitudinal centre plane of an excavation is a plane of symmetry and not subjected to shear stress. This situation sometimes raises a point of discussion as to the need to install tendons in the centre of a roadway. There are several reasons for why it may be beneficial to install centre tendons at or close to the centre of a roadway. These include:

- Centre tendons assist strata to span between adjacent ground support elements and provide a degree of protection against falls of ground between these elements.
- Although centre tendons may not offer direct resistance to shear, they indirectly do resist shear if they are sufficiently long to anchor above the zone of sagging roof strata. Figure 7.5 shows an extreme example of how sag results in parting planes between beds, with upper beds ultimately bridging across the excavation. Sufficiently long tendons function to not only suspend the sagged strata but, if installed early enough, to resist deflection of the immediate roof strata. This feature can be very important when the roof is subjected to lateral load since it reduces the eccentricity of the loading system, thereby increasing the critical load required to cause the roof to buckle, Fig. 2.49. Restricting deflection is also important for limiting strata deformation and for maintaining clear access for men, equipment and ventilation.
- A centre tendon anchored into an upper competent bed does not, strictly speaking, result in a halving of the effective span of an excavation. Nevertheless, depending on tendon density and stiffness, a centre tendon can result in a significant reduction in effective span. If the ideal outcome of a halving of the span could be achieved then, as reference to the 2.75 m roadway width case in Table 7.6 shows, this would result in roof deflection being reduced to only 6 %, bending stress to 25 %, and shear stress to 50 % of the comparative values for a 5.5 m roadway, as well as a fourfold increase in the critical load required to induce buckling.

A second point of discussion which also arises at times is the need and merits of fully encapsulating a centre bolt, especially when it is not subjected to shear. As is typical in ground engineering, there is no unique answer; each situation has to be assessed on its own merits. Reasons for fully encapsulating a centre tendon include:

- it is a stiffer support system and therefore provides greater resistance to deflection;
- in some circumstances, especially in the vicinity of one or more goaf edges, a centre tendon may be subjected to shear during its life cycle;
- if the tendon breaks or the collar assembly fails, a portion of the tendon still has a capacity to resist dilation; and
- it removes its susceptibility to unloading as a result of damage and rock crushing at the collar.

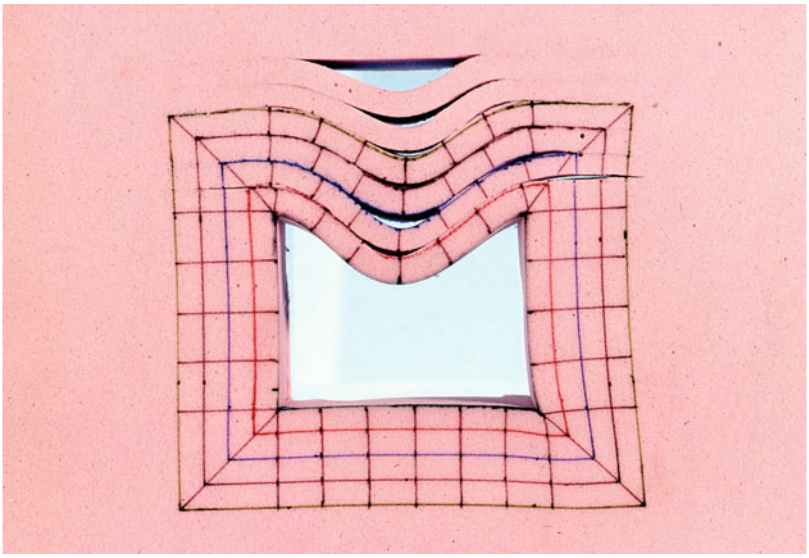


**Fig. 7.5** Extreme illustration of shear between beds and the development of partings in sagging roof strata, ultimately resulting in a doming out against a bridging bed.

Conversely, in situations where displacement cannot be resisted, it may be beneficial not to fully encapsulate a centre tendon, or to delay full encapsulation as in the case reported by Payne (2008). Delaying full encapsulation allows for a certain amount of cable stretch, or strata relaxation, with the objective of providing 'temporary' roof support until the inherent strength of the dilated rock mass has been mobilised. It also enables operators to gauge the state of loading from the condition of the face plate assembly. Subsequent full encapsulation increases the stiffness of the tendon and, therefore, its resistance to further displacement.

An example drawn from practice illustrates this principle. Consider an 8 m long cable with a load transfer rate of 800 kN/m (80 t/m), a yield load capacity of 600 kN (60 t) at 1 % elongation, and an elongation at ultimate failure of 6 %. If the upper 2 m of this cable were to be fully embedded, elongation of the unencapsulated portion would be 60 mm at yield and 360 mm at breakage. If the same cable were to be fully encapsulated, load transfer across a parting at yield would be distributed over a length of 1.5 m, giving an elongation at yield of about 15 mm. The same elongation in the 6 m free end of the end anchored cable would give a strain of 15/6000 = 0.25 %, at which point the resistance provided by the cable to bed separation would be only 150 kN (15 t). Hence, the end anchored cable can sustain some 60 mm of displacement before going into yield as compared to 15 mm for the fully encapsulated cable. If the cable were to be fully encapsulated after 15 mm of elongation, it could still sustain at least another 11 mm of elongation before going into yield, depending on whether the parting develops in the initial end anchored section of the cable or in its remaining section.

### 7.5.4 Effectiveness of Pretension

The merits of pretensioning a tendon is another topic of debate. Often, an opinion is premised on field performance observations, with little regard to the mechanics of behaviour applicable to the situation. There is a range of mechanistic based reasons for why pretensioning may deliver substantial benefits in some situations and not in others. These can relate to:

- The location of partings relative to the anchorage horizon of the tendon (see Sect. 6.4.3).
- The relative contribution of pretension to the key factors that influence tendon performance in a given environment. For example, in the case study presented in Sect. 6.4.2 and summarised in Fig. 6.29, joint roughness is the primary determinant of resistance to shear displacement (consistent with Eq. 2.25).
- Deformation may be driven primarily by a mechanism that is relatively insensitive to pretensioning.
- The drivers of deformation may be sufficient to overwhelm pretensioning.

### 7.5.5 Stress Relief

A range of factors impact on the lateral stress field around an excavation and can be manipulated as stress control measures to either restrict the development of horizontal stress or to reduce an existing stress level. These factors include:

- orientation of the mine layout;
- sequence of mining;
- sequence of secondary extraction;
- pillar and barrier width;
- excavation height;
- yield pillars;
- formation of stress relief slots;
- sacrificial roadways;
- reinforcement type, pattern and density.

Situations can also arise where mining results in a reduction in vertical stress. These are related mostly to multiseam operations and tend to be of a regional nature. However, vertical stress relief can occur on a site-specific basis in situations such as after back-holing a roadway (see Sect. 5.2.3); after a total extraction panel has retreated through a double stress notch (see Sect. 5.2.4);

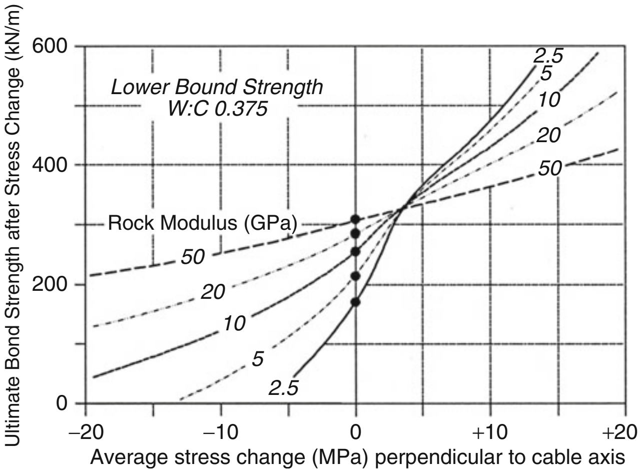


**Fig. 7.6** An example of the effect of stress change and rock modulus on bond strength (After Hutchinson and Diederichs 1996).

and after a periodic weighting event in a total extraction panel (see Sect. 9.5.4).

Research has shown that stress change in the surrounding rock mass after the installation of a tendon can significantly affect the shear strength of the tendon-rock interface (see for example, Hutchinson and Diederichs 1996, and Offner 2000). Stress relaxation reduces confinement and, therefore, the shear strength of the interface, while stress increase can have the opposite effect as shown in Fig. 7.6. Water make from roof strata is often a sign that at least one of the principal stresses is tensile. It should always be treated with suspicion and evaluated accordingly.

### 7.5.6 Coal Roof

In some environments, particularly where the immediate roof is predominantly coal, the roof can be quite pliable and may converge in excess of 50 mm without displaying obvious signs such as centreline cracking, guttering, or abnormal rib spall. This displacement can be difficult to detect without the aid of instrumentation.

A particularly treacherous situation arises if the coal roof is prone to shear failure at the abutments. In some cases, the propensity for shear failure may be increased due to a reduction in lateral stress in the coal roof as a result of de-gassing and de-watering. Experience shows that in these circumstances, instrumentation (other than that which monitors and alarms in real time) may not always give adequate warning of impending failure. In these environments, careful consideration needs to be given to the orientation of the mine workings relative to cleat and joint direction, to mining span, and to the routine installation of long tendons at intersections. Notably, experience highlights the importance of operators detecting and responding to the subtlest of physical signs of change in the workplace. In some cases, a very small localised spalling of a coal rib, particularly towards the top of the ribside, has proven to be a reliable precursor to a fall of ground in this type of environment. In other environments, this type of subtle change would hold no significance and is likely to go unnoticed.

### 7.5.7 Floor

It needs to be borne in mind that the floor strata may also have poor physical integrity and be subjected to high lateral stress concentrations, particularly when negotiating a geologically disturbed area. This can have indirect but significant consequences for roof control because poor floor conditions can impede face advance rate, aggravate rib conditions and lead to an increase in span, and result in a loss in bearing capacity. Concreting and rock bolting have proven to be effective controls in some mines.

### 7.5.8 Monitoring at Height

Although ground engineers conceptualise and discuss ground control predominantly in terms of stress, in practice decision making is based predominantly on observations of displacement, which includes observations of the state of roof, ribs, floor and goaves. Physical inspections by the workforce are the primary source of frequent and regular observation. The rigour of these inspections decreases in high workings if the inspections are made from ground level, simply because displacement cannot be reliably detected by the naked eye in these situations. Therefore, when forming high working places that cannot be accessed at roof level for inspection, consideration should be given to the installation of remote monitoring and to the availability of lighting sources that are more powerful than cap lamps.

### 7.5.9 Mining Through Cross Measures

If the mining horizon is not parallel to strata bedding, a series of free surfaces or ‘lips’ is created at the roof and floor horizons. These situations are often referred to as ‘mining through cross measures’ and are most commonly associated with developing declines or drifts to access a coal seam. However, they can also occur during mining operations when, for example, mining is taking place on a cross grade; an excavation has to made into the roof to accommodate equipment or to facilitate ventilation; there is a change in mining elevation due to faulting; or the cutting head sumps into the roof.

When a cut is made into the roof to increase the mining height, the resulting step in the roof profile is referred to as a ‘brow’. Mining through cross measures is of concern because it effectively creates one or more cantilevered beams. In the case of long drivages, such as a drift, the systematic support pattern is designed to take account of this situation. Otherwise, consideration needs to be given to developing support rules for specific situations. Brows have been implicated in many serious and fatal falls of ground in underground coal mining and always warrant special attention. The appropriate support pattern is a site-specific matter but, in general, support should be installed within 0.5 m of the lip of a brow, at regular intervals along the whole length of the brow. Depending on local conditions, the lip and face of the brow may also need to be restrained with mesh.

The sumping of the cutter head into the roof creates a series of small brows which can present an elevated risk of falls of brat through to falls of large slabs of rock. If a weak parting plane is present at the roof horizon, consideration should be given to undercutting this plane and allowing the remaining top coal to fall to it.

## 7.6 Rib Control

### 7.6.1 Introduction

In underground coal mining, the sidewall of an excavation is known as the rib, **ribside** or **ribline**. Ribs are the interface between coal pillar behaviour and excavation behaviour. Rib control is particularly important for reasons of safety, pillar stability, roof control and productivity. It takes on added significance with trends towards greater depths of mining and more confined workplaces due to a combination of reduction in roadway width (as a strata control measure) and increase in the size of mining equipment.

Deterioration of coal pillar ribs is generally referred to as **spalling**, **scaling**, or **sloughing**. It has four primary sources, which may be interactive, these being displacement on geological discontinuities; material failure due to mining-induced stress; weathering upon exposure to the mine atmosphere; and disintegration in the presence of water.

Ribs can be very complex structures to analyse and control. This is because they can comprise a number of horizontal beds, of different structural and mechanical properties, interspersed with dirt bands that can have material properties ranging from that of soils to that of rocks, with these beds being imprinted with at least two sub-vertical joint sets and then subjected to a mining-induced change in stress that induces deformation and further fracturing. Hence, coal seam fabric, geological structure, the mechanical response of the pillar system to seam compression, and weathering in the presence of air and water are important considerations when developing rib control strategies. These strategies need to give consideration to both structurally controlled instability and to stress controlled instability, which can be interactive.

Historically, coal ribs used to be supported primarily by short lengths of timber, or **sprags**, wedged between the rib and adjacent timber props used to support the roof and to secure the ventilation curtain that directed air to and from the mining face. With the demise of timber props, rib support systems have evolved mostly from adapting roof support technology on a trial and error basis. Essentially, rib support and reinforcement have been based on roof support concepts and hardware that have been rotated through 90°, with limited attempts to understand the mechanics of rib deformation, load transfer and response to reinforcement elements (Hebblewhite et al. 1998).

### 7.6.2 Risk Profile

Ribs are a hazard that present a serious risk to safety. In Australia, the NSW underground coal sector has a comprehensive incident database going back many decades. This shows that falls of rib have historically accounted for nearly 60 % of lost time injuries (LTI) and 50 % of fatalities associated with falls of ground in NSW. The severity of injuries, as measured by the duration of lost time injuries, is higher for falls of rib than for falls of roof. Rib spall also contributes to other types of lost time injuries, such as slips and trips and back injuries due to jarring as mobile equipment travels over rib spall. Mark et al. (2008) reported that rib falls injure approximately 100 mine workers annually in the USA and accounted for 15 fatalities in the preceding 13 years. All four fatalities due to falls of ground in the Australian underground coal mining sector between 2006 and 2014 were associated with ribs, of which one was a double fatality related to a pressure burst.

Although rib fall incidents still account for 50–60 % of fall of ground related injuries in NSW, the number of rib related injuries has decreased by over an order of magnitude in the last two decades (Fig. 1.6). The main reasons for this improvement have been the transition from hand held bolting to machine mounted bolting rigs fitted with protective shields and temporary rib support systems and the increased use and density of rib support, including full meshing.

### 7.6.3 Rib Composition

Coal is an organic material, with different phases in plant growth being reflected in the plies that make up a coal seam. Significant differences in composition and structure can occur between coal plies, as evidenced by the development of an Australian Standard specific to distinguishing and sampling coal plies (AS-2519-1993 1993). This standard provides a framework for classifying a coal ply under one of five categories on the basis of its brightness which, in turn, is an indicator of the concentration of the maceral 'vitrinite'.

Bright coal plies generally contain a higher density of jointing than dull plies. In theory, ply specific joints are defined to be **cleats**, as distinct from joints that may persist through the full seam thickness and even into the immediate roof and floor. In practice, however, the distinction between cleats and joints has become blurred, particularly since cleats in dull coal seams can extend through the full seam thickness.

Cleating is usually comprised of two orthogonal joint sets, one of which is generally dominant and known as **face cleat** and the other less well developed and known as **butt cleat**. These cleat planes are often polished or mineralised. It is not uncommon for cleat direction to vary between 10° and 20° at a site and to rotate up to 40° or more across a mining lease.

Based on the concept that a brightness rating infers a measure of volumetric cleat density, Medhurst and Brown (1996, 1998) developed a

**Table 7.7** Coal ply classification scheme devised by Medhurst and Brown (1996) on the basis of AS-2519-1993 (1993)

| Category | Description | Scale |
|---|---|---|
| B (C1) | Bright | >90 % bright |
| Bd (C2) | Bright banded | 60–90 % bright |
| DB (C3) | Interbanded bright and dull | 40–60 % bright |
| Db (C4) | Dull with minor bright | 10–40 % bright |
| Dmb (C5) | Dull banded | 1–10 % bright |
| D (C6) | Dull | <1 % bright |

six category classification scheme with the aim of capturing the compositional and structural features that most influence the mechanical response of a coal seam. This scheme, shown in Table 7.7, was premised on the five category brightness scheme of AS-2519-1993 (1993). The researchers concluded on the basis of their scheme and a large scale laboratory testing of coal samples, that their results indicated different brightness category coals have different strength and deformability characteristics. Furthermore, duller coals exhibit strong brittle behaviour while brighter coals have a much softer response to loading, reflecting the effect of the higher density of discontinuities.

In addition to the presence and relative location of coal plies and their associated cleat directions and densities, the fabric of a rib is also determined by features such as the thickness and composition of in-seam stone bands; roof, floor and in-seam interfaces; the relative locations of stone bands and interfaces; the presence of roof and floor stone due to floor ‘grubbing’ or roof ‘brushing’; joint density, direction and dip; and mining-induced fractures. Hence, a variety of rib responses to mining can be expected.

### 7.6.4 Rib Behaviour

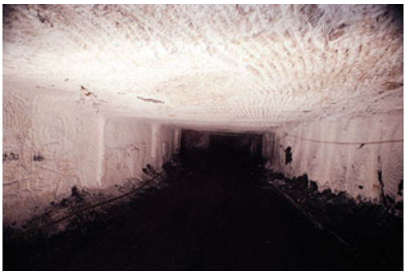
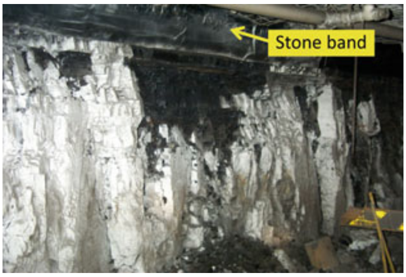
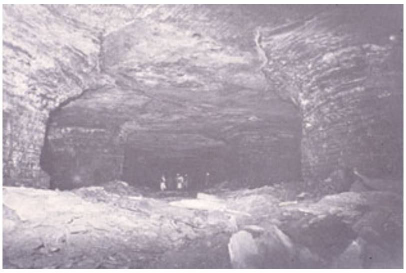
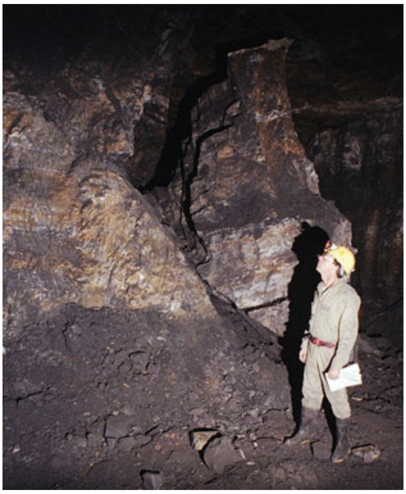
When an excavation is formed:

- Ribs are afforded the opportunity to spall and to slide or topple into the mining void under the effect of gravity, irrespective of any mining-induced changes in stress field and state of fracturing.
- The normal stress acting on the riblines is reduced to zero and the tangential stress become a multiple of the pre-mining stress, thereby generating a step increase in deviator stress ($\sigma_1$–$\sigma_3$) at the ribline. This is conducive to mining-induced fracturing and dilation of the ribs.
- Mining-induced fracturing is likely to increase the severity of rib spall because it trends near parallel to the ribsides.
- The opportunity is provided for low cohesion and friction in-seam bands and roof and floor interfaces to extrude into the excavation and, therefore, induce lateral tension in the ribsides. Lateral tension can generate new fractures and mobilise existing fractures, thus aggravating rib spall such as in the cases shown in Figs. 4.14, 4.15 and 4.19.

Table 7.8, complemented with Figs. 4.13, 4.14, 4.15, 4.19 and 4.32 illustrates some of the behaviour modes of coal pillar ribs. It is readily apparent from this suite of photos that there is a wide range of rib behaviour, both on a regional scale between sites and on a very localised scale over the height of an individual coal rib. The photographs illustrate that unless it is decided to install a blanket support system comprising long tendons and steel mesh, rib support design must always give careful consideration to site-specific conditions.

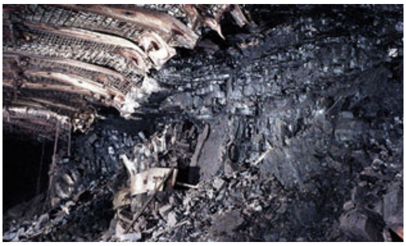
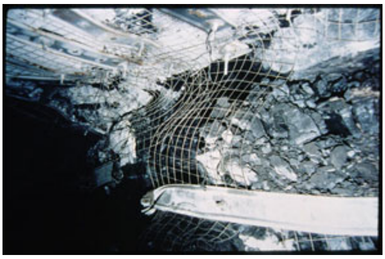
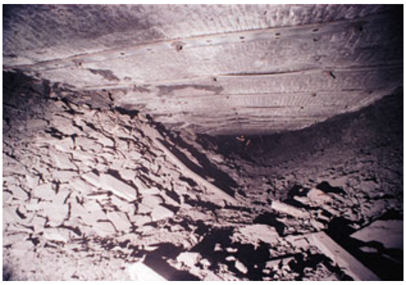
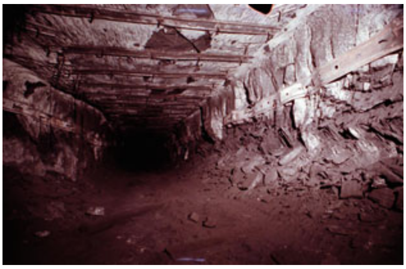
In homogenous material, mining-induced fracturing aligns with stress trajectories (or streamlines), resulting in ribsides reflecting the influence of end confinement and taking on an hourglass shape. This is observed in coal ribs to varying degrees, depending on the fabric of the coal and the stress field (see Fig. 4.13 for example). Gates et al. (2008) reported that pillars

**Table 7.8** Examples of the range of rib conditions encountered in underground coal mining

| | |
|---|---|
|  | (a) Ribs formed by a single pass continuous miner in an ‘as-cut’ (unsupported) condition at shallow depth in a dominantly dull banded coal seam. |
|  | (b) A hazardous situation for safety due to rib spall in a well jointed seam undermining a mudstone band in the top portion of the ribline. |
|  | (c) High ribsides in excess of 100 years old at shallow depth |
|  | (d) Rib spall on a well defined inclined joint (or **slip**) in a pillar more than 4 m high, presenting a higher risk of injury from a fall of ground and resulting in loss of much of the load bearing area of the pillar. (Anderson) |

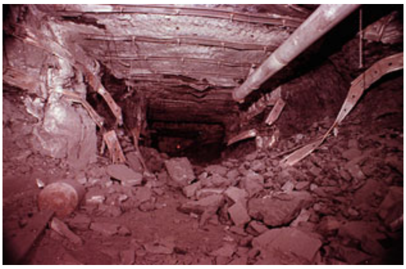
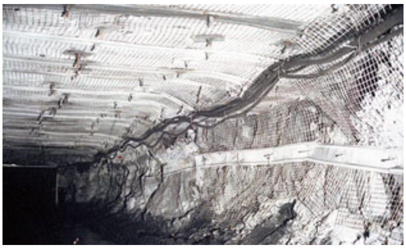
(continued)

**Table 7.8** (continued)

| | |
|---|---|
|  | (e) Unravelling on development of very highly cleated ribs supported with fibreglass bolts and plywood straps at a depth of approximately 200 m. Many of the bolts were sheared and rib spall resulted in more than a 2 m, or 40 %, increase in roadway width. Mine management debated whether the ribs should be supported before the roof in order to limit the adverse effects of rib spall on roof span. |
|  | (f) Steel mesh, W straps and bolts used in an attempt to confine ribs in the conditions shown in photograph (e). |
|  | (g) Rib spall in a roadway close to the abutment of a pillar extraction goaf. |
|  | (h) Rib spall concentrated in a bright coal ply in the lower portion of a pillar, below the W straps. |

(continued)

**Table 7.8** (continued)

|  |  |
|---|---|
|  | (i) Condition of ribs on the outbye side of a barrier pillar at the end of a longwall panel at a depth of approximately 500 m. |
|  | (j) Rib spall arrested by bolting, strapping and fully meshing the roof and ribs with steel products at the face on development. |

associated with the Crandall Canyon Mine pillar failure incident exhibited an hourglass profile. In practice, however, hourglass rib profiles are not the norm in coal seams because of the over-riding influence on rib spall of the intensity, dip and vertical persistence of joints; in-seam interfaces; and rock mass strength variations over the height of a rib. The method of extraction, mining direction, excavation shape, extraction sequence, and rate of extraction can also influence the shape of the rib profile.

The net effect of coal seam fabric is that ribs can be comprised of blocky material of any size, ranging from small particles through to slabs and columns that may weigh several tonnes, with the structural and mechanical properties of these blocks varying over the height of the ribs and with the direction of mining. While in theory, the weakest portion of a rib is at its mid-height, in practice, this can occur at any height in the rib side.

Pervasive cleats and joints give rise to hazards in their own right, especially when they trend in much the same direction as a ribside. These planes of weakness may define large vertical slabs which can slide or topple from the ribside without warning. The risk is elevated in the presence of slippery or extrusive interfaces and when dip is associated with the joints. A very small amount of movement on an interface can be sufficient to remove restraint to a slab, while dip increases the likelihood that joints will daylight in the ribside and, therefore, enable blocks to slide or topple from the rib.

Ribs on one side of a roadway can take on a different appearance to those on the other side of the roadway when joint sets dip. Sometimes, ribs are prone to spall on one side of an excavation by sliding and on the other side by toppling. In some cases, one side of an excavation may be impacted to a greater extent than the other. O’Beirne et al. (1987) reported, for example, that at Huntley Colliery in New Zealand, about 72 % of roadway cross-sections displayed left-hand to right-hand side differences when cleat dip was less than 85°, compared to 36 % at higher dip angles. When dip was less than 70°, rib failures at the top on one side of a roadway and at the bottom on the other side of the roadway were common.

The effect of mining-induced stress on rib stability can be evaluated in the first instance by applying simple empirical pillar strength formulations, such as those presented in

Chap. 4. These produce a uniaxial strength range for a one metre cube of coal of typically 6–8 MPa, corresponding to a pre-mining cover load of 240–320 m. The formation of an excavation increases the resultant load on a pillar, with this load increase concentrated at the pillar edges until failure occurs, when the load is then transferred further into the pillar. An areal extraction of 50 %, for example, results in an average pillar stress of 6 MPa at a depth of 120 m, and 8 MPa at a depth of 160 m. Hence, at mining depths in excess of about 200 m, it is unlikely that failure of the coal ribs can be prevented. The objective of support then becomes one of minimising post-failure deformation of the coal in the yield zone and maintaining the integrity of the yield zone.

The effect of pillar compression on rib behaviour is well demonstrated by the in situ coal pillar test measurements of Wagner (1974), shown in Fig. 4.20. Mining height determines rib stiffness and the extent to which the mid-height of the rib experiences the beneficial confinement generated at the roof and floor contacts. The higher the ribs, the less confinement provided to the mid-height of the pillar and, therefore, the greater the depth of mining-induced fracturing and rib spall. Roof and floor contact conditions and interfaces have a significant influence on the rate at which this confinement is generated. Pillar confinement builds up more slowly in weaker roof and floor strata and in the presence of low shear strength interfaces.

Further insight into rib behaviour is provided by the analytical analysis of Salamon (1995b) that supports the advanced confined core concept, discussed in Sect. 4.4.5. Yielding rib coal softens as it is strained and its cohesion and strength diminish. If unconfined, it disintegrates when its cohesion is lost, to become ‘slumped’ coal (Fig. 4.22). If confined, it eventually becomes ‘crushed’ coal. Crushed coal has no cohesion or residual unconfined compressive strength and can only survive in this state if it is supported or ‘confined’. Salamon (1995b) deduced from analytical modelling of highly stressed coal pillars (see Sect. 4.4.5) that a situation could arise where the elastic and yielding zones could be suddenly transformed into four zones with the potential for this process to be violent and to result in a pressure burst.

The support provided to the roof by a rib is reduced over the rib yield zone and virtually non-existent over the crush zone, thus resulting in an increase in effective roof span and the potential for roof failure to develop over the ribside itself. This can be extremely hazardous since shear failure and guttering may develop in the roof above the yield or crush zone of the rib, without even being visible within the roadway until extensive and uncontrollable instability has developed (Salamon 1995).

Very friable ribs, which are prone to unravel if not confined, and crushed and yielded ribsides are not amenable to being supported with conventional roof support systems. Research by Hebblewhite et al. (1998) identified the very localised and limited influence of a stiff reinforcement element in soft and weak material, with the development of relatively high bolt loads but an inability to achieve effective load transfer from the bolt to the surrounding coal beyond a very small radius of influence. Apart from during the early stages of deformation, it can be very difficult to reinforce friable and crushed ribs with conventional bolting systems, with load transfer not being maintained during the later stages of loading. A bolt acts as a rigid inclusion, with the coal failing around it and leaving the bolt ineffectual as a reinforcing tendon. The only contribution that the bolt can make at this stage is to assist to some degree in holding mesh against the rib, provided that the head plate assembly is still functional.

Techniques to aid in managing the risks of rib fall in these situations include anchoring long tendons into the elastic zone of the pillar as a means for securing a rib surface support system, such as mesh and wrapping cables around pillars, with or without a complementary surface support system. In both cases, it is desirable that the support system has a capacity to yield in order to provide sustained support to the ribs. Yielding support systems also provide enhanced protection from dynamic instability events such as pressure bursts and are used extensively in the underground metalliferous sector for these purposes.

In summary, there is no single mode of rib behaviour and behaviour mode can change throughout the life cycle of an excavation or pillar. Therefore, support systems may be required to perform different functions over time, depending on the nature of ground displacements, stage of mining, and mode of pillar behaviour. This requires that careful consideration be given to site-specific conditions over the full life cycle of the excavation. For these reasons, rib support design is not a statistical or desktop exercise and no one design procedure can be expected to cover all situations.

### 7.6.5 Design Considerations

The following matters have the potential to impact on rib control and, therefore, warrant careful consideration when designing mine layouts:

- Cleat and joint density, dip and direction need to be determined early in the design process. In the case of a greenfield site, this determination may have to be based solely on borehole core and associated geophysical testing. In a brownfield site, consideration should be given to also obtaining structural mapping information from adjacent mines, albeit that these parameters can change over distance.
- Design should integrate joint and cleat density with information concerning the location and nature of coal plies and the mechanical properties of the interfaces between these plies, including soil or rock bands within the seam.
- Experience suggests that orientating excavations at least 20°, and preferably 25–30°, to the major and minor cleat and joint directions can be effective in mitigating the risk of rib falls. However, it may result in an elevated risk of large blocks falling out of the rib at pillar corners. Additional support measures, such as strapping or meshing may need to be implemented at pillar corners to mitigate this risk.
- In the case of pillar extraction, it is almost inevitable that, at some stage in the mining operation, cleat direction will be near parallel to ribsides that cannot be safely accessed and supported. This stage should coincide with the least time exposure to mining in that direction and the manner and sequence of extracting pillars should be designed so that operators are never required to work beside unsupported ribs (irrespective of the relative direction of the cleat).
- An assessment should be made of how mining-induced fracturing is likely to interact with natural fracturing. It may be closely aligned or it may provide an extra degree of freedom for movement and define slabs and wedges which can topple or slide.
- Factors such as the dip direction of joints and the dip of the coal seam can result in different rib behaviours on either side of an excavation. This might require a different support response for each side of the excavation or, preferably from a practical perspective, one support response that caters for both types of behaviour.

- Rib stability becomes more problematic as mining height increases. There is a higher likelihood that discontinuities will daylight in the ribsides, that friable ribs will unravel, and that columns and slabs of rib will buckle. Mining-induced fracturing is likely to extend further into the pillar at mid-height due to a reduction or loss in the beneficial effects of end constraint. Rib falls present a greater risk of injury due to the greater height from which they can originate. In a study of Australian mines with the highest incidents of rib fall related injuries in the period 1986/1987–1992/1993, Fabjanczyk and Guy (1994) found that the height of the rib contributed significantly more to rib fall-related injuries than did the depth of mining. These factors may require a change in rib support strategy from that employed at lower mining heights.
- Single pass continuous miners cause considerably less damage to ribs than multiple-pass

machines and can enhance rib stability considerably.
- There are benefits in installing rib support as close as possible to the face. In some situations, the ribs may have already failed ahead of the mining face. In others, the ribs may unravel immediately upon exposure. Early intervention can be as important, or more important, for roof control than for rib control. McCowan (1994) reported that, although a difficult concept to understand by many, rib control was viewed as far more effective than roof reinforcement in controlling the roof at Ellalong Colliery (Australia), where rib failure always preceded roof failure. Galvin (1996b) also reported that soft ribs were associated with all roof falls at Angus Place Colliery (Australia) at the time. The application of beam theory provides some perspective to these experiences, with one metre of rib spall on each side of a 5 m wide roadway having the potential to result in almost a fourfold increase in the deflection of thin roof beds.
- While each situation needs to be assessed on a site-specific basis, in general, dull bands should be targeted as horizons for installing tendons and used to anchor and retain cross supports and/or mesh systems over bright bands.
- In pressure burst environments, consideration should be given to the use of a yielding form of rib support, preferably integrated with a flexible, full surface coverage support element.

### 7.6.6 Support Hardware Considerations

Experience demonstrates that the quality of rib support installations installed with hand-held bolters is inferior to installations installed with machine mounted rigs. Hand-held bolters also expose operators to rib spall and manual handling risks and limit the height at which rib support systems can be installed. In stressed and friable conditions, it is difficult to manually drill a hole longer than 1.5 m due to the effort required to push the drill into the hole and to retract the drill rods.

In longwall mining, a range of limitations are associated with installing metal rib support systems on the longwall block side of roadways. These relate to:

- incendiary sparking when the shearer cutter picks strike the support;
- flying projectiles when the shearer cutter picks strike the support;
- tendons and straps becoming wrapped around the shearer cutter head;
- torn conveyor belts from sharp pieces of the metal support system;
- blockage of conveyor chutes and transfer points by metal support elements.;
- operator exposure to rib falls while endeavouring to recover the support elements before they enter the coal transport system.

In an effort to overcome these limitations, a range of cuttable fibreglass and plastic based support elements have been introduced. While beneficial, none are yet to deliver the performance of metal support elements. Of note is that:

- fibreglass bolts have a high tensile strength but poor elongation characteristics, a low tolerance to bending and a low shear strength. Consequently, they rupture at about one-tenth of the elongation sustainable by steel and are particularly prone to break in shear when a lower section of the rib spalls.
- Plastic bolts have a much lower shear strength than fibreglass bolts and are susceptible to creep.

In friable, crushed or high abutment stress conditions, it is common for a rib bolt hole to close as the drill steel is withdrawn. Self-drilling bolts have the potential to overcome this problem, provided that the encapsulation medium can be made to extrude into the hole annulus. This is not always possible.

Mechanically end anchored bolts can find application in these circumstances because the diameter of the drill hole is considerably greater

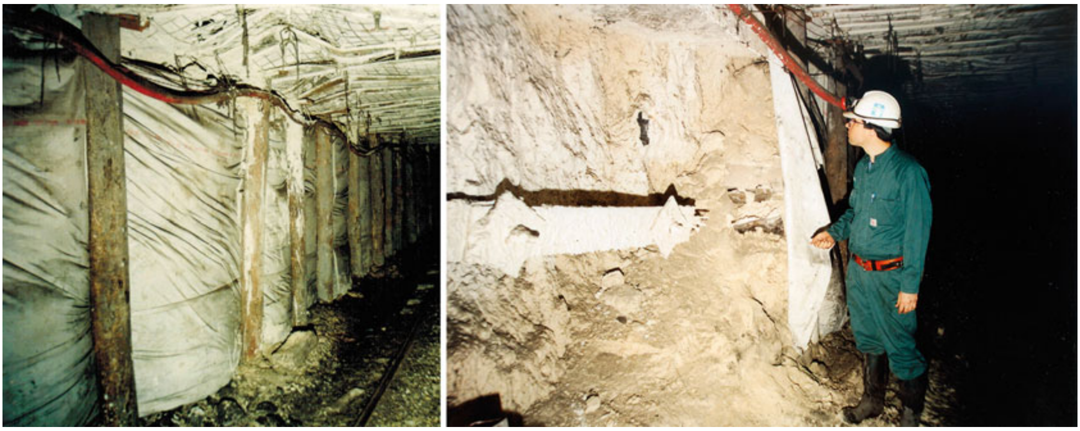


**Fig. 7.7** An example of rib replacement to mitigate the effects of severe rib spall of the type shown in Table 7.8e (After Galvin 1996).

than that of the bolt in order to accommodate the mechanical anchor. However, their use is still premised on being able to pass the mechanical anchor down the hole. Mechanically end anchored bolts also find application as a form of yielding support.

In situations of severe rib spall, rib replacement has proven beneficial (Fig. 7.7). This involves backfilling a ribline with a low strength monolithic grout that re-establishes confinement to the rib.

### 7.6.7 Operational Considerations

Operational considerations specific to coal ribs include:

- It is important to regularly review cleat direction and to modify rib support systems or, if need be and possible, the mine layout to manage the impacts of change in cleat direction.
- A major change in cleat direction over a short distance (typically, 10–50 m) can be indicative of a geological structure in the vicinity and/or of a significant change in stress field and/or ground conditions. Hence, it is judicious to monitor for change in cleat direction and to include it as a trigger in ground control Trigger Action Response Plans.
- It is much more difficult, if not impossible at times, to support ribs once mining infrastructure such as conveyor belts and monorails has been installed. Therefore, careful consideration needs to be given to installing all secondary rib support at the time of initial drivage.
- Personnel need to be alert to the impact of spall concentrated in the lower plies of a rib. It is often the case that it is not the spalled material that constitutes the risk but the remaining undercut strata, which can include unsupported roof.
- When roadways are aligned so as not to be sub-parallel to cleat, there is an elevated risk of rib spall at pillar corners. Support systems installed on pillar corners may need to be anchored several metres back from the corners in order to be effective in holding a corner in place and not be dislodged as part of a rib fall.
- Support installed on pillar corners is vulnerable to damage from mobile plant. Controls need to be implemented to prevent personnel from being injured by a rib fall as a consequence of machinery impacting a pillar corner.
- All damaged rib support in areas where personnel are exposed to the risk of a rib fall, and in areas where it is critical that rib support continues to remain effective, should be rectified immediately.
- Steel-based rib support systems present a risk of injury from razor sharp steel edges and

burrs if they suffer impact damage. Torn W straps, in particular, present an elevated risk of injury. Inspection and monitoring schemes need to make provision for removing this risk.

- In addition to safety and serviceability, rib control is important for reasons of productivity and cost. A workplace free of loose coal reduces the problems associated with pumping slurries; avoids the formation of wheel ruts and their implications for slip, trips and equipment damage; and improves wheeling times for coal transport vehicles.
- Spalled coal can provide substantial confinement to coal pillars in old workings. There are anecdotal reports of pillar collapse being initiated when 'slack' coal (rib spall) was recovered from old, small pillar workings at shallow depth.

### 7.6.8 Summary Conclusions

Rib stability is a serious hazard in the underground coal mining sector and, therefore, requires diligent risk management. Factors that impact on rib stability include coal fabric, depth, mining height, roof and floor contact conditions, in-seam bands, and mining-induced stress. Given the range in these factors, the various combinations in which they may come together and interact, and the propensity for them to vary significantly across a mining lease, rib support design is not a purely statistical or desktop exercise. Careful and regular consideration must be given to the nature and implications of site-specific conditions for rib stability. Rib behaviour should be a critical element in ground control management systems, such as Ground Control Management Plans and the Trigger Action Response Plans that support these plans.

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# 8 Pillar Extraction

**Abstract**

Pillar extraction, (also referred to as retreat mining*;* pillar recovery*;* stooping*;* pillar robbing*;* and bord and pillar second workings) is the practice of forming a series of pillars and then partially or totally extracting some or all of the pillars, usually with mining operations retreating out of a panel. There are a number of basic pillar extraction methods, with numerous permutations being associated with some of these methods. The practice has a history of being the most hazardous form of underground coal mining and a reputation for being an art as much as a science. However, since the mid 1990s, research into coal pillar mechanics in combination with new technology, education and a risk management approach to design and operation, have resulted in substantial improvements in the safety of pillar extraction.

This chapter commences with defining terminology specific to pillar extraction and reviewing attributes of the mining system that account for it being such a hazardous form of secondary coal extraction. It then presents a range of mining layouts and extraction sequences that have been developed in endeavours to reduce risk related to loss of ground control. This forms the foundation for evaluating ground control under the headings of global stability; panel stability; and workplace stability. Consideration is given to factors such as panel width to depth ratio; shape and age of pillars; goaf edge behaviour; goaf edge support systems; the design and impact of remnant pillars; and the significance of speed of extraction. It concludes by emphasising the importance of having operational discipline.

**Keywords**

Abutment stress • ARMPS • Bord and pillar second workings • Breaker line • Breaker prop • Christmas tree method • Fender • Finger line • Goaf • Hydraulic mining • Intersections • Lifting left and right • Local stability • Longwall mining • Manner and sequence of extraction • Mobile breaker line support • Mobile roof support • Multiple seam workings • Munmorah method • Old Ben method • Open ended lifting • Operating practices •

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Panel and pillar • Partial pillar extraction • Pillar extraction methods • Pillar load • Pillar robbing • Pocket and wing • Pressure burst • Pushout • Regional stability • Retreat mining • Rib pillar • Safety factor • Shortwall mining • Snook • Split and lift • Spontaneous combustion • Stook • Stooping • System stiffness • Treetopping • Twinning • Ventilation • Wongawilli method

## 8.1 Introduction

**Pillar extraction**, (also referred to as **retreat mining; pillar recovery; stooping; pillar robbing**; and **bord and pillar second workings**) is the practice of forming a series of pillars and then partially or totally extracting some or all of the pillars, usually as mining operations retreat out of the panel. There are a number of basic pillar extraction techniques and numerous variations of these. The practice has a history of being the most hazardous form of underground coal mining and a reputation for being an art as much as a science, evolving through trial and error, observation and experience. However, since the mid 1990s, research into coal pillar mechanics in combination with new technology, education and risk management approaches to design and operation have resulted in substantial improvements in safety in pillar extraction.

## 8.2 Attributes of Pillar Extraction

In the eleven years to 1992, 15 fatalities due to falls of ground occurred during pillar extraction operations in NSW, Australia, of which 12 were associated with buried continuous miners (Galvin 1993c). One incident was a triple fatality. Between 1989 and 1992 alone, there were 44 incidents of continuous miners being trapped by falls of ground in pillar extraction operations in NSW for periods exceeding seven hours (this being the statutory criterion for reporting these incidents to the regulator). At the time, some 95 % of the machines had on-board drivers and 57 % of the falls buried the driver's cab (Galvin 1994a). There were numerous near misses that went unreported because machines were recovered in less than seven hours.

A similar situation was associated with pillar extraction in the USA at the time. Although pillar extraction accounted for only some 10 % of underground coal production between 1989 and 1996, 25 % of the 111 fatalities due to falls of roof and rib in this period were attributable to pillar extraction (Mark et al. 1997). In the decade 1992–2001, falls of ground in pillar extraction resulted in 27 fatalities, with persons in pillar extraction panels reported to be three times more likely to be fatally injured than those in other panels (Mark et al. 2003).

The two largest mining disasters in the Australian mining industry since 1984 (Moura No. 4 Colliery – 12 fatalities, Moura No. 2 Colliery – 11 fatalities) and the most serious near-miss event (Endeavour Colliery – 8 escaped) were all associated with explosions in pillar extraction panels. In each case, summarised in Appendix 9, inquiries identified deficiencies in panel design as a contributing factor, reflecting that ground engineers must also give careful consideration to managing the risks of spontaneous combustion, frictional ignition and gas explosion when designing mine layouts.

One of the most notable factors that cause pillar extraction to be so hazardous is that, although it is conducted on the retreat on a panel basis, it is not a true retreat operation at the coal face. This is because pillars are extracted by a series of cuts towards the goaf and, thus in the opposite direction to panel retreat (Fig. 8.1a). This exposes the workplace to falls of ground by progressively removing ground support from the area through which operations have to subsequently retreat and by creating a fulcrum that encourages caving to develop outbye of the working face.

The risk of a fall of ground is elevated because multiple headings and cut-throughs are required

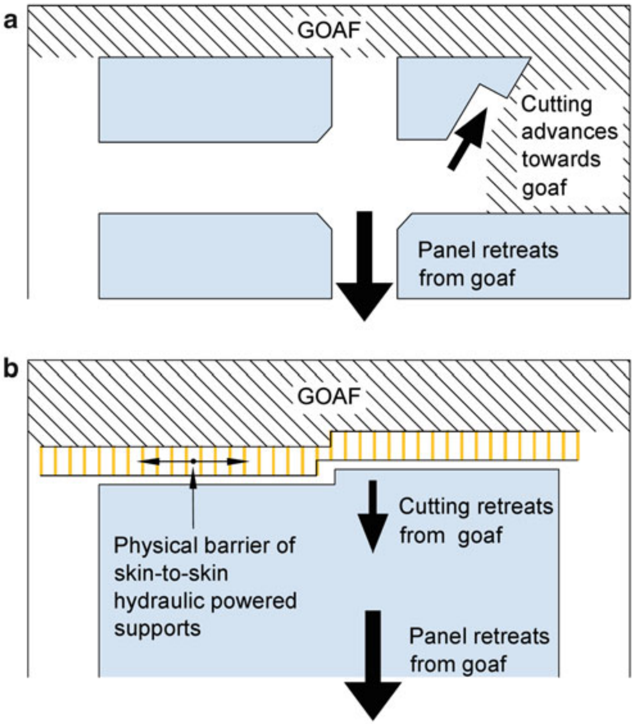

a
GOAF
Cutting advances towards goaf
Panel retreats from goaf
b
GOAF
Cutting retreats from goaf
Physical barrier of skin-to-skin hydraulic powered supports
Panel retreats from goaf

**Fig. 8.1** Illustration of some of the significant differences between (**a**) pillar extraction on the retreat and (**b**) longwall mining on the retreat

to service the working face and these are located in the abutment stress zone. Every time extraction of a pillar is complete, operations have to retreat through a heading or cut-through, analogous to driving a longwall face into a pre-developed roadway. The situation is particularly hazardous as effective roof spans are greater due to the presence of intersections; the intersections have had time to deteriorate; and the roadways cannot be obstructed with standing support or permitted to converge excessively as they are travel ways for mobile equipment and personnel. Geological features elevate the high risk of a fall of ground in the workplace and can change goafing behaviour. A change in geological conditions, even on a very local scale, can require a change in the manner and sequence of extracting a pillar.

Hence, operations are constantly exposed to the goaf, with operators sometimes required to work close to the goaf edge or otherwise position themselves back from the goaf edge and, therefore, in the zone of high abutment stress. Unlike in longwall mining, there are no inbuilt controls on excavation dimensions; the method is not amenable to a high level of automation; the excavation size does not increase in small, uniform incremental steps; the area exposed by mining is not immediately supported; mining does not constantly retreat away from the goaf; and operators and operations do not have the benefit of working under the protection of an engineered physical barrier (Fig. 8.1b).

A number of aspects of pillar extraction elevate the risk of spontaneous combustion and gas explosion. Invariably, a considerable amount of broken coal remains in the goaf in the form of windrows, rib spall, and crushed remnant pillars. This provides a potential heating source and fuel supply for spontaneous combustion. Remnant pillars can impede caving and compaction of the goaf, providing an opportunity for sufficient oxygen ingress to support spontaneous combustion in the goaf but insufficient air flow to remove the heat generated by this oxidation process. These attributes were associated with the Moura No. 2 Colliery disaster, summarised in Appendix 9. Depending on the panel layout, even relatively small goaf falls in partial or total pillar extraction layouts can generate large windblasts that, depending on the panel layout, can displace flammable and explosive mixtures throughout the mine, as in the case of the Moura No. 4 Colliery and Endeavour Colliery explosions.

Historically, these attributes of pillar extraction resulted in safety becoming highly reliant on operators observing and responding subjectively to signs of strata instability, such as *the roof working, the roof dribbling, the floor bumping, the goaf working, goaf talk, rib spall, timber props taking weight,* and *props cracking*. It became the norm in some cases for operations to be regularly *chased out* by these conditions (reference for example, Shepherd and Lewandowski 1994).

Following five fatalities in pillar extraction in three separate incidents in NSW in the early 1990s, the management of ground control risk in pillar extraction came in for critical review. An overseas study of pillar extraction practices was undertaken (Galvin et al. 1991), a pillar extraction manual was developed (MDG-1005 1992), and a major industry funded research program with a focus on industry education and technology transfer was established at the University of New South Wales (Galvin et al. 1994; Galvin 1996; Hebblewhite et al. 1996). The USA also embarked on a number of initiatives to improve ground control in pillar extraction, including the development of the (ARMPS) procedure for dimensioning coal pillars (Mark and Chase 1997).

Subsequently, there have been step improvements in safety in pillar extraction. In the 21 years to 2012, there were five fatalities in pillar extraction in Australia, three due to falls of roof and two to falls of rib. A sixth person was revived after being buried by a rib fall. Significantly, three of these incidents were associated with attempting to reach remote controlled continuous miners immobilised under unsupported roof and beside unsupported ribs and a fourth with over-extraction using a remote controlled continuous miner. These incidents highlight the need to carefully assess what new risks may be associated with a control introduced to manage an identified risk.

During the four years to 2010, one fatal roof fall occurred during pillar extraction in the USA, compared to an average of two per year during the previous decade (NIOSH 2010). Six mine workers and three rescue personnel were also killed in 2007 as a result of coal pillar failure initiated by a pressure burst during pillar extraction at Crandall Canyon Mine. Prior to this, there had been only one fatal pressure burst during pillar extraction in the USA since the mid 1980s. A pressure burst in 2013 and another in 2014 claimed a further three lives during pillar extraction (see Sect. 11.8 for a more in-depth discussion on pressure bursts).

In summary, fatalities due to falls of ground in pillar extraction operations have decreased by some six-fold in Australia and some 16-fold in the USA since the early to mid 1990s. The main contributors to these improvements have been advances in:

- Geotechnical knowledge. A better understanding of pillar mechanics and excavation behaviour, supported by advances in numerical modelling, has resulted in safer and more productive mine designs and ground support systems.
- Mining equipment technology. The introduction of remote controlled continuous miners and mobile breaker line supports (MBLS), or mobile roof supports (MRS), has reduced operator exposure to the working face, goaf falls and manual handling hazards.
- Ground support technology and practices. Improved ground support systems provide for a more secure working place at the goaf edge and in the abutment stress zone.
- Risk assessment. Risk assessment consistent with International Standard ISO 31000 *Risk Management* (ISO 31000 2009) and supported with guidelines such as New South Wales Mining Design Guidelines MDG-1005 – *Pillar Extraction Manual* (MDG-1005 1992) and MDG 1010 – *Risk Management Guideline* (MDG-1010 2011) have proven very effective when embedded into all aspects of pillar extraction including mine design, equipment selection, support selection, operating procedures, and responding to change.

- Consultation and education. The practical experience of operators is invaluable in understanding strata behaviour in pillar extraction. Conversely, educating operators in basic strata control principles promotes an understanding of the need to comply with pillar extraction designs and an improved capability to recognise design inadequacies and opportunities for improvements.
- Operating discipline. There is a higher awareness of the need for and enforcement of compliance with mine design and reduced latitude for operators to vary extraction sequences without consulting senior management.

These advances are interactive and essential elements of risk management. The most significant benefits have been derived from integrating them to devise safer methods and sequences for extracting coal pillars.

## 8.3 Basic Pillar Extraction Techniques

### 8.3.1 Design and Support Terminology

There is a range of terminology and jargon specific to pillar extraction, the meaning of which can vary between countries. Figures 8.2 and 8.3 illustrate the terminology adopted in this text, with the more basic terms defined in Table 8.1 in the approximate order in which they are encountered in pillar extraction processes.

Figure 8.3 shows the extensive use of two rows of timber props set at the goaf edge to encourage the roof to break off at that point and to prevent a goaf fall extending into the working face. These props are referred to as **breaker props.** Demanding transport and manual handling requirements can be associated with setting them. Furthermore, they are particularly prone to being dislodged in a roof fall. For these and other reasons, breaker props have been replaced in many collieries by so-called **mobile roof supports**, or **MRS**. The first MRS were manufactured by Voest Alpine and employed at Middelbult Colliery in South Africa in 1984. Known as Alpine Breaker Line Supports (ABLS), they were trialled in Australia at Cooranbong and Nebo Collieries in 1987 and quickly gained wide acceptance and application. A photograph of an early model ABLS is shown in Fig. 8.4. MRS were introduced in the USA in 1988, with these units being manufactured by Fletcher and known as Fletcher Mobile Roof Supports (FMRS).

While there are differences between an ABLS and a FMRS, both types of MRS essentially comprise a roof canopy supported by four hydraulic yieldable legs mounted on a crawler track frame. The canopy has a capacity for transverse and longitudinal displacement in order to conform to the roof horizon but is stabilised against scissor collapse in these directions by some form of mechanical linkage system, such as a lemniscate linkage (discussed in Sect. 9.3). Protection against goaf flushing is provided by chain curtains. The area of the canopy can range from about 3.5–8 $m^2$, with dimensions of 4 m × 2 m being typical in seam heights greater than 2.5 m.

Support capacities range from 4 MN to 7.3 MN (400–730 t), with the front legs and back legs operating as independent units. Vertical stiffness ranges between 214 kN/mm and 300 kN/mm (21.4 t/mm and 30 t/mm) (Barczak and Gearhart 1997) compared to just over 30 kN/mm (3 t/mm) for an Australian hardwood prop (Shepherd and Lewandowski 1994). The stiffness of a MRS unit decreases with increase in the number of leg stages due to the associated reduction in cylinder cross-sectional area. The machines are fitted with a plough used to clear debris as they are advanced and to lift the front crawlers over obstacles. They are powered electrically by a trailing cable and designed to be operated by remote control during an extraction cycle, umbilical cord during tramming, and manually during maintenance. Indications of leg pressure and rate of rise in leg pressure can be provided on large dials or by coloured light units that can be seen from some distance.

As an aid to understanding some pillar extraction layouts, Fig. 8.3 also shows a **finger line**. This support practice comprises a row of props installed in a lift immediately after its completion, to assist operators in monitoring

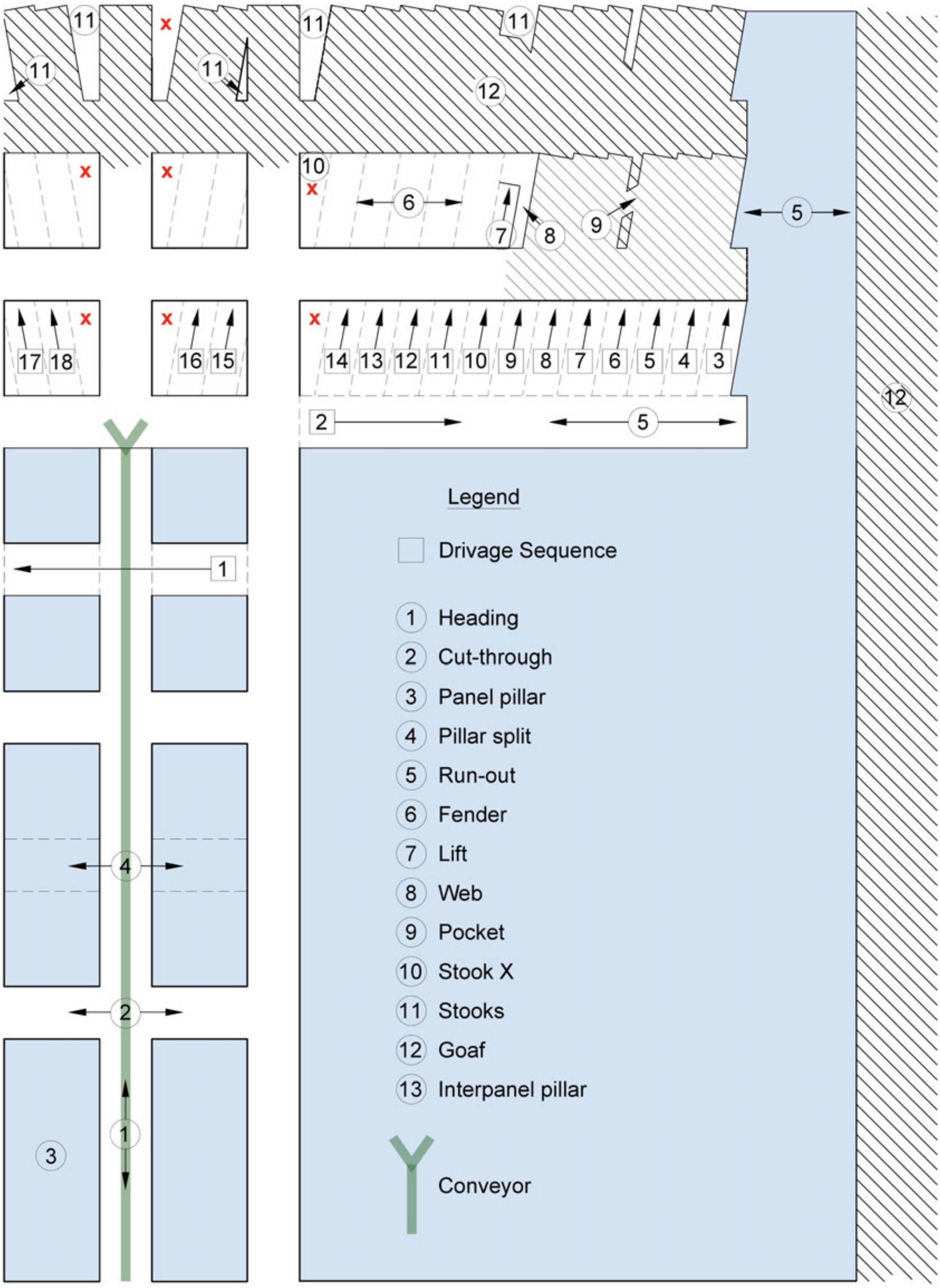

11
x
11
11
11
12
11
x
x
10
x
6
7
8
9
5
17 18
x
x
16 15
x
14 13 12 11 10 9 8 7 6 5 4 3
2
5
12
1
4
2
1
3
Legend
Drivage Sequence
1 Heading
2 Cut-through
3 Panel pillar
4 Pillar split
5 Run-out
6 Fender
7 Lift
8 Web
9 Pocket
10 Stook X
11 Stooks
12 Goaf
13 Interpanel pillar
Conveyor

**Fig. 8.2** Terminology relating to pillar extraction layouts

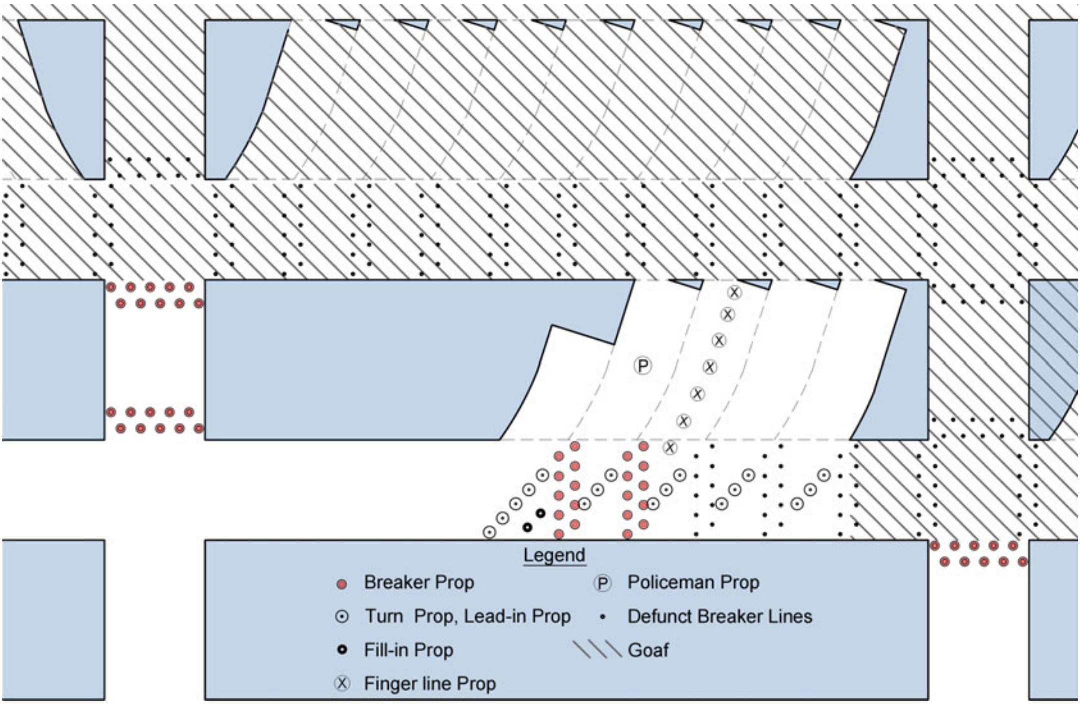


**Fig. 8.3** Terminology relating to support in pillar extraction (not all props in the goaf are shown)

**Table 8.1** Basic pillar extraction terminology adopted in this text

| | |
|---|---|
| Split | A roadway developed within a pillar to divide it into smaller portions. May also be referred to as a **pocket**. |
| Run-out | A long split driven from the main development to the flanks of a pillar extraction panel. |
| Lift | A slice of coal mined from a pillar for the purpose of extracting the pillar. A lift may be mined from a heading, cut-through or split. |
| Fender | A long rectangular or slender web of coal separating a split or lift from the goaf. Also referred to as a **wing** or a **web** in some situations. A fender may or may not be subsequently extracted, or only partially extracted. |
| Web | A thin fender of coal left between two lifts, usually as a temporary support measure. Portions of a web may be extracted (pocketed) on retreat out of a lift. |
| Stook | A term used in Australian pillar extraction operations to describe a remnant portion of a pillar not extracted. When this stook is adjacent to the last lift in a pillar, it is referred to as a **snook** in South Africa and as a **stump** or a **pushout** in the USA, the extraction of which is not necessarily prohibited in these two countries. |
| Stook X | A remnant portion of a pillar that is not permitted to be extracted in Australian pillar extraction operations. It always includes remnant coal adjacent (outbye) to the last lift and often remnant coal inbye of the first lift in a pillar. |
| Stripping | The process of reducing the size of a pillar by mining lifts from its perimeter. Also referred to as **slabbing**. |
| Sequence | The order in which pillars are developed and/or lifted off. |
| MRS | A mobile roof support, which includes both Voest Alpine Mobile Breaker Line Supports (MBLS) and Fletcher Mobilised Roof Supports (FMRS). |

convergence in the goaf and to provide some degree of protection from local roof falls during extraction of the next adjacent lift. A combination of regulatory restrictions; corporate policies that prohibit employees from venturing out under unsupported roof; and the introduction of remote controlled continuous miners, MRS and risk management approaches, has resulted in the use of finger line props being discontinued in many organisations.

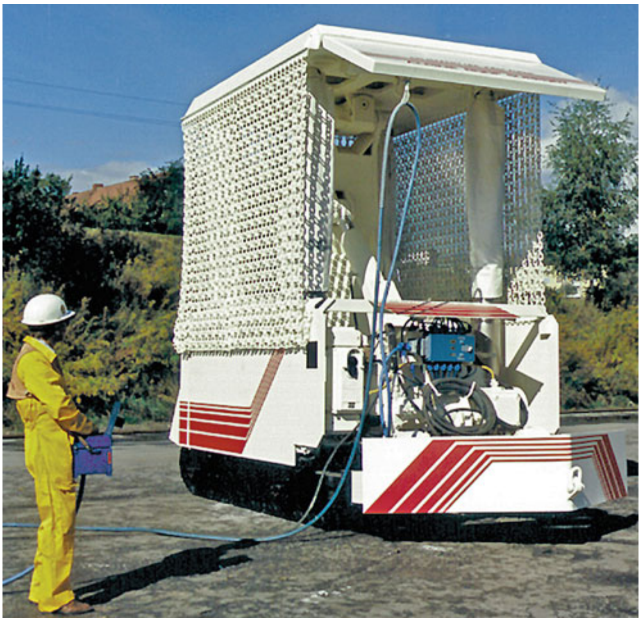


**Fig. 8.4** A Voest Alpine mobile breaker line support (Copyright: Sandvik Mining)

## 8.3.2 Total Extraction Methods

The wide range of basic mining methods for extracting coal pillars and the numerous variations within these methods is illustrated, for example, by the review of 34 mines in Kentucky (USA) undertaken by Marshall Miller and Associates (2006). This identified 165 different pillar extraction plans. Factors which impact on choice of method and sequence of extraction include whether the pillars were originally designed to be extracted; seam thickness; seam dip and dip direction; cleat direction and density; in situ stress field; strength and structural integrity of the superincumbent and floor strata; gas regime; propensity for spontaneous combustion; availability of capital to support equipment acquisition; type of continuous miner; type of haulage system; experience of the workforce; subsurface subsidence constraints; surface subsidence constraints; and regulatory requirements. Only the basic extraction methods are presented in this chapter. Examples of the numerous variants are to be found in literature. While the use of some of these methods has declined significantly, they are presented because they assist in providing an understanding of the foundation principles that underpin pillar extraction and in avoiding mistakes of the past.

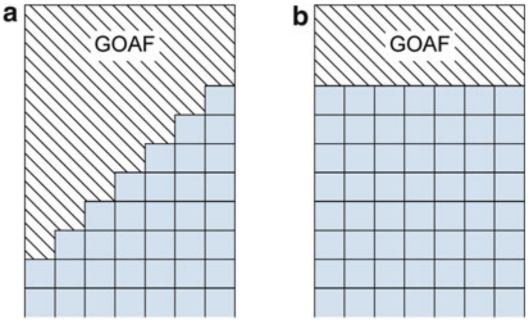


**Fig. 8.5** Generic layouts for extracting standing pillars. (**a**) 45° extraction line. (**b**) 90° extraction line

Pillar extraction methods can be classified under two generic categories, namely:

- Extraction of a panel of pillars pre-formed during primary development. These pillars are referred to as **standing pillars** (Fig. 8.5).
- Extraction of pillars mostly formed as part of a secondary extraction process. These pillars are referred to as **green pillars** (Fig. 8.6).

It has been the practice when extracting standing pillars in some countries to orientate the extraction line at 45° to the direction of retreat (Fig. 8.5a). This can offer a number of ground control benefits because the acute end of the extraction line is better protected from abutment stress, large goaf spans, and goaf ingress; while if the obtuse end is on the virgin block side of the panel, it is subjected to conditions that, generally, are not much different to those associated with the face when it is orientated at 90° to the retreat direction (Fig. 8.5b). Coal haulage and conveyor belt side-loading considerations associated with mechanisation have resulted in the demise of this practice.

Extraction of standing coal pillars can present an elevated risk because of the large number of intersections and because ground conditions have had time to deteriorate. Elevated risk can be associated with pillars not designed with subsequent pillar extraction in mind, as this can result in suboptimal layouts and dimensions for pillar extraction.

The risks associated with intersections and deterioration in ground conditions over time have been addressed with pillar extraction layouts that minimise primary development, intersections and standing pillars in the process of accessing the inbye end of the panel. The concept is shown in Fig. 8.6a. Operations then retreat out of the panel by driving sequences of one of more long splits to one side of the panel and immediately extracting the associated 'green' pillars or fenders on retreat back to the main development. In order to maximise the extraction to development ratio, some variations involve forming extraction workings on both sides of the main development on retreat from the panel (Fig. 8.6b). Ground engineering aspects that influence the selection of a pillar extraction technique are discussed in more detail in Sect. 8.4.

### 8.3.2.1 Conventional Methods

**Open ended lifting** (or **skirting**) is one of the earliest forms of pillar extraction and the most hazardous. The pillar is progressively reduced in size by mining lifts that may be up to 30 m long off the perimeter of the pillar, thereby effectively placing operators in the goaf (Fig. 8.7). Variants include setting one or two rows of 'breaker' props along the external flank of each lift and leaving a thin fender of coal between the goaf and each lift. The death of five operators in an incident involving open-ended lifting in NSW in 1966 (Galvin 1996) led to the demise of the method in Australia in favour of conventional split and lifting methods, the Wongawilli method and the Old Ben method of pillar extraction (all of which are explained later in this Section). In mature risk management cultures, there is limited potential for open-ended lifting today, even when utilising remote controlled machinery.

**Diagonal splitting** involves extracting a pillar by mining a series of diagonal lifts partially or totally through the pillar. Some examples of this

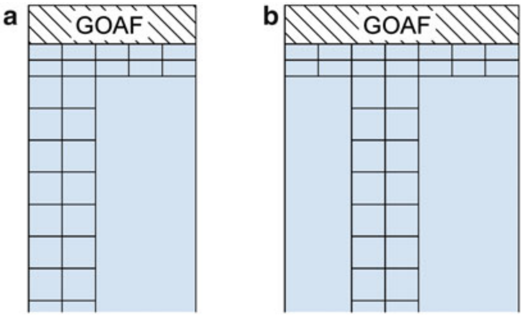


**Fig. 8.6** Generic layouts for extracting green pillars. (**a**) Single side extraction. (**b**) Double sided extraction

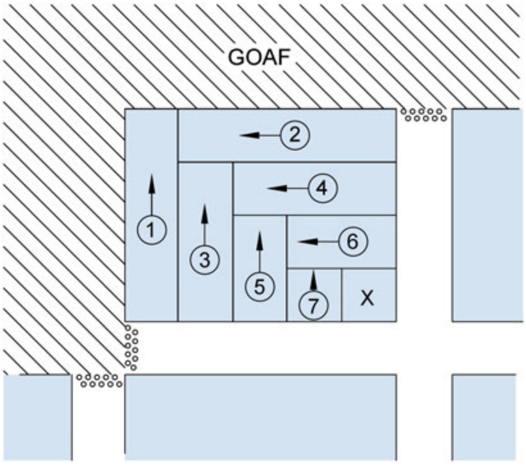


**Fig. 8.7** The most general form of open ended lifting (or skirting), with lifts being segregated by rows of props or a coal web in some variants

technique are illustrated in Fig. 8.8. It is characterised by taking alternate lifts from the outside of the pillar, so as to work towards the core of the pillar. This is intended to remove yielded coal with the lowest load carrying capacity first and leave the higher strength 'green' coal in a confined state for as long as possible. The method is suited to smaller pillars and has found application in South Africa and the USA, where pillar size can range down to around 6 m square. Legislation restricts minimum pillar width in Australia to 10 m and the method has not found application in that country.

**Split and lift** circumvents many of the risks associated with open ended lifting by driving one or more splits into the pillar to form substantially wide fenders that are then lifted off on the retreat (Fig. 8.9). There are many variants of the method and the terminology for describing these. In this text, 'split and lift' includes pocket and wing and pocket and fender. The splits have to be supported on development and equipment usually has to negotiate a 90° turn at the entrance to the split. Both these aspects slow down operations and provide additional time for ground conditions to deteriorate under high abutment stress.

'T' intersections can also be a serious impediment to being able to rapidly withdraw the continuous miner and MRS units after the last lift has been mined from a fender or if ground conditions deteriorate. This is sometimes addressed by pre-splitting the adjacent outbye pillar to form a four way intersection to provide direct access to the face. This approach requires particular care because it results in smaller pillars and more intersections that are subjected to high abutment stress for an extended period of time.

A popular variant of split and lift pillar extraction in the USA was the so-called **Old Ben system**, shown in Fig. 8.10a. This found extensive use in strong roof conditions in Australia from the 1960s to around 2000. It was modified as shown in Fig. 8.10b by adding pillars on the return side of the panel and moving away from a 'straight' goaf line, to become known as **Modified Old Ben** or **Munmorah method** of pillar extraction (Hanrahan 1993). Panel width ranged from 100 to 250 m, with 160–180 m being typical. The method requires good roof and floor conditions and facilitates good face ventilation. As depth of mining increases, abutment loading can cause problems because much of the working area falls within the abutment stress zone. Hence, its use in Australia has been limited to depths less than around 250 m.

The **Wongawilli method** of pillar extraction evolved in Australia in the late 1950s (Sleeman 1993) and involved driving and supporting very long splits off the main development headings to form fenders some 7–9 m wide. These were then lifted off on retreat together with the main development pillars (Fig. 8.11). It was believed at the time that the split was driven in destressed or 'winded' coal (Grant 1993), although some now question if this was always the case. Ventilation and wheeling constraints typically limited the length of run-outs to 60–80 m. The method was attractive because it minimised the number of intersections; provided a very repetitive method of operation that removed operator discretion to change the sequence of mining; and facilitated uniformity between production shifts.

Disadvantages associated with the Wongawilli method include susceptibility to off-centre drivage; a lack of ventilation at the coal face in long splits; egress impeded by deteriorating ground conditions; and long haulage distances between shunts (passing points) when mining the run-outs. Off-centre drivage towards the goaf side can weaken the fender, while if towards the solid, it can place operations in a higher stress zone as well as increase the length of lifts. Holing the fender at some point along its length in order to maintain adequate ventilation at the coal face creates an additional intersection that can present stability risks during retreat out of the split. Rate of mining is reduced significantly towards the end of run outs due to time lost waiting for haulage vehicles to shunt outbye and travel long distances.

Up until longwall mining was proven technically and economically viable in Australia in the early 1980s, the Wongawilli method dominated secondary extraction at depths greater than about 250 m in that country. In particular, it was suited

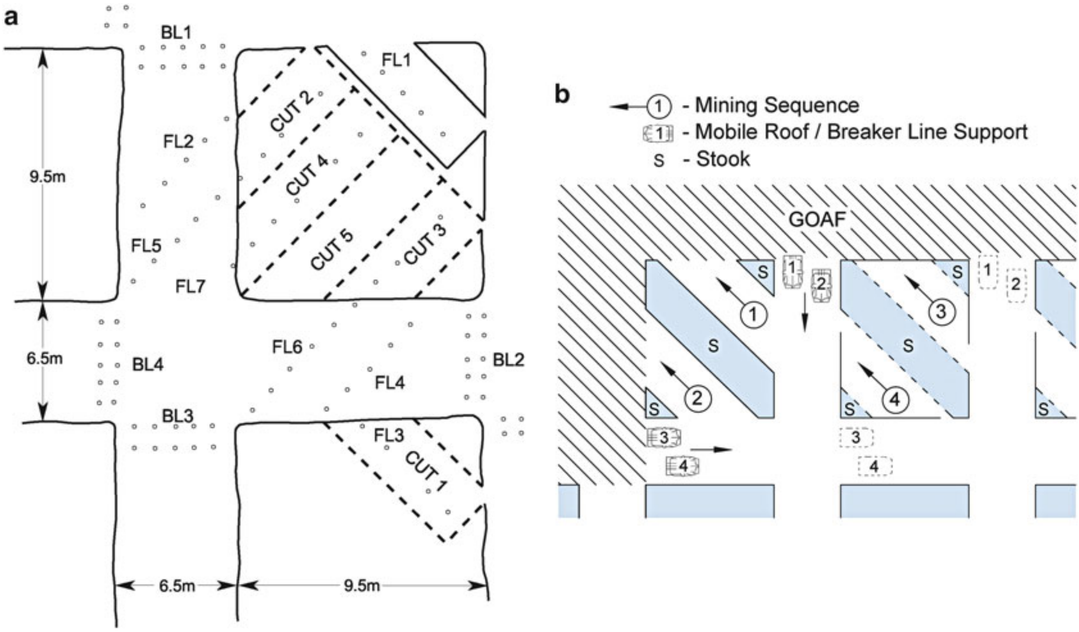


**Fig. 8.8** Examples of methods and sequences for extracting pillars using diagonal lifts. (**a**) (Beukes 1990). (**b**) (Galvin 1993b)

to cover depths down to 600 m in the Southern Coalfield of NSW, where all other methods of pillar extraction had failed (Sleeman 1993). The method is also suited to shallower depths and continues to find application in a range of modified forms.

A number of hybrid pillar extraction methods based on the Old Ben and the Wongawilli methods have been developed with the aim of providing a better balance between development and extraction. A noted example is rib pillar extraction utilising two continuous miners as shown in Fig. 8.12, one deployed on development and the other on extraction.

If a continuous miner has an on-board operator, the length of a lift needs to be restricted to about 6 m if the operator is not to venture under unsupported roof. In order to maximise fender width for stability purposes while also maximising coal recovery, lifts have to be driven as close as possible to 90° to the split. This is another impediment to achieving a rapid rate of extraction and to being able to rapidly retreat back under supported roof. It contributes to most pillar extraction methods that use on-board continuous miner operators being characterised by a low extraction to development ratio.

Nearly all pillar extraction in Australia, South Africa and the USA is now undertaken with remote controlled continuous miners, enabling extended lifts of up to 15 m before the haulage operator becomes exposed to unsupported roof. Lifts can also be driven at a flatter angle, typically 60–75°, due to the extra reach of the continuous miner. These features provide the opportunity to lift off both sides of a split during retreat and facilitate better goaf control (because less remnant coal is left in the goaf) and faster extraction and egress out of a lift, while keeping operators away from the goaf edge.

In Australia and South Africa, lifting off both sides of a split is referred to as lifting **left and right**. In the USA, the practice is referred to variously as **left-right**, **twinning**, **Christmas tree mining**, **fishbone** or **treetopping**, an example of which is shown in Fig. 8.13. Lifting off only one side of a roadway is referred to as **single sided lifting** in Australia and as **outside lifting** in the USA. Lifting off practices differ between the

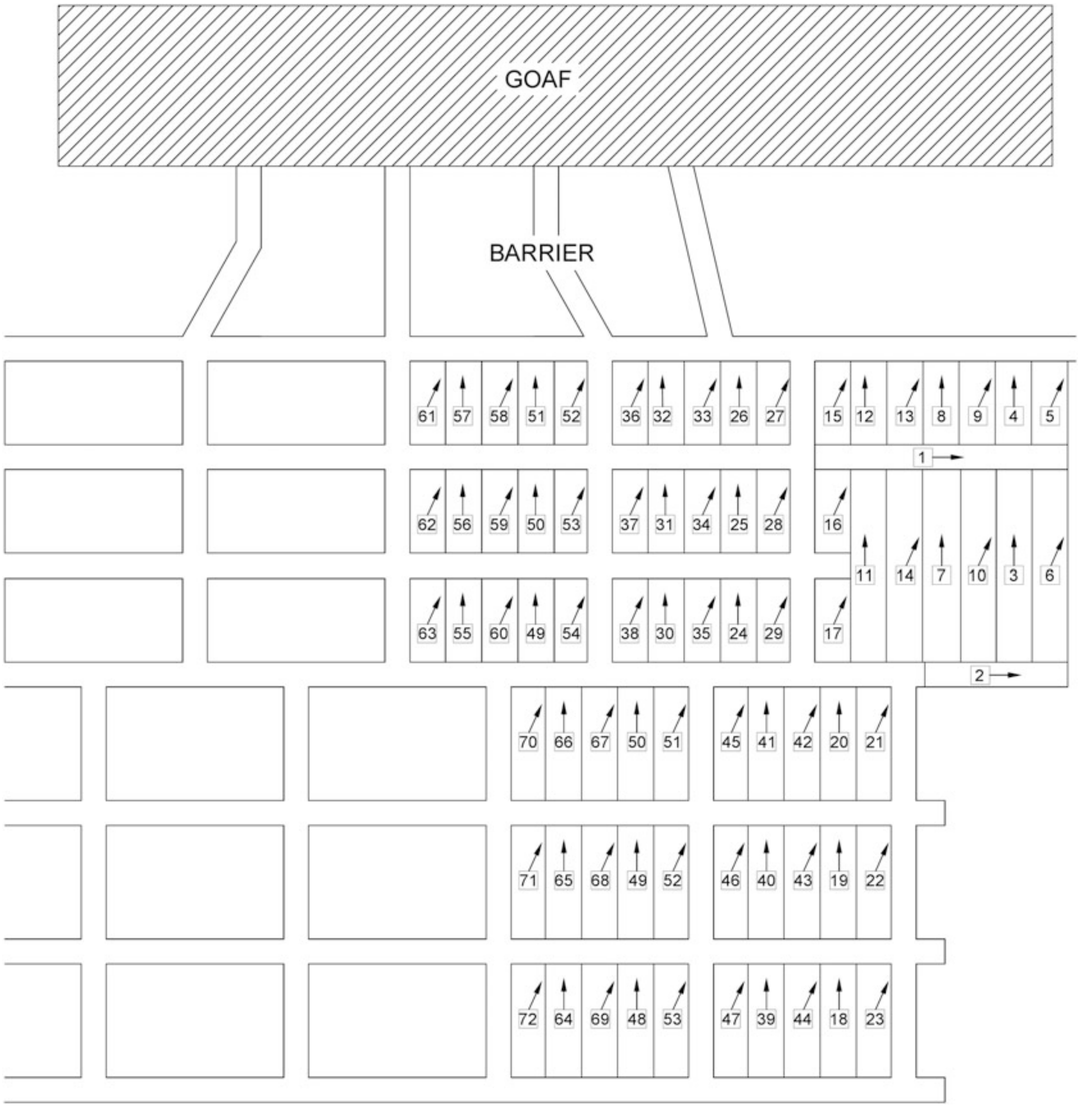

GOAF
BARRIER
61
57
58
51
52
36
32
33
26
27
15
12
13
8
9
4
5
1
62
56
59
50
53
37
31
34
25
28
16
11
14
7
10
3
6
63
55
60
49
54
38
30
35
24
29
17
2
70
66
67
50
51
45
41
42
20
21
71
65
68
49
52
46
40
43
19
22
72
64
69
48
53
47
39
44
18
23

**Fig. 8.9** An example of split and lift pillar extraction

two countries in that super units (two continuous miners) and place changing are used in the USA, with the latter resulting in two pillars being extracted simultaneously, while these practices are not adopted in Australia. Australian practice is to use three MRS units at the face when lifting off left and right and to stagger lifts (Fig. 8.14), whereas USA practice is to employ two MRS units and generally not stagger lifts. Additional MRS units are employed at intersections in the USA whereas timber or rock bolt breaker lines may be relied upon in Australia.

A number of other changes to methods and manners for extracting coal pillars have arisen out of the introduction of remote controlled continuous miners and MRS. MRS have been found to be susceptible to structural damage when operating under strong roof that cantilevers a considerable distance out into the goaf before caving. This prompted a shift in Australia away

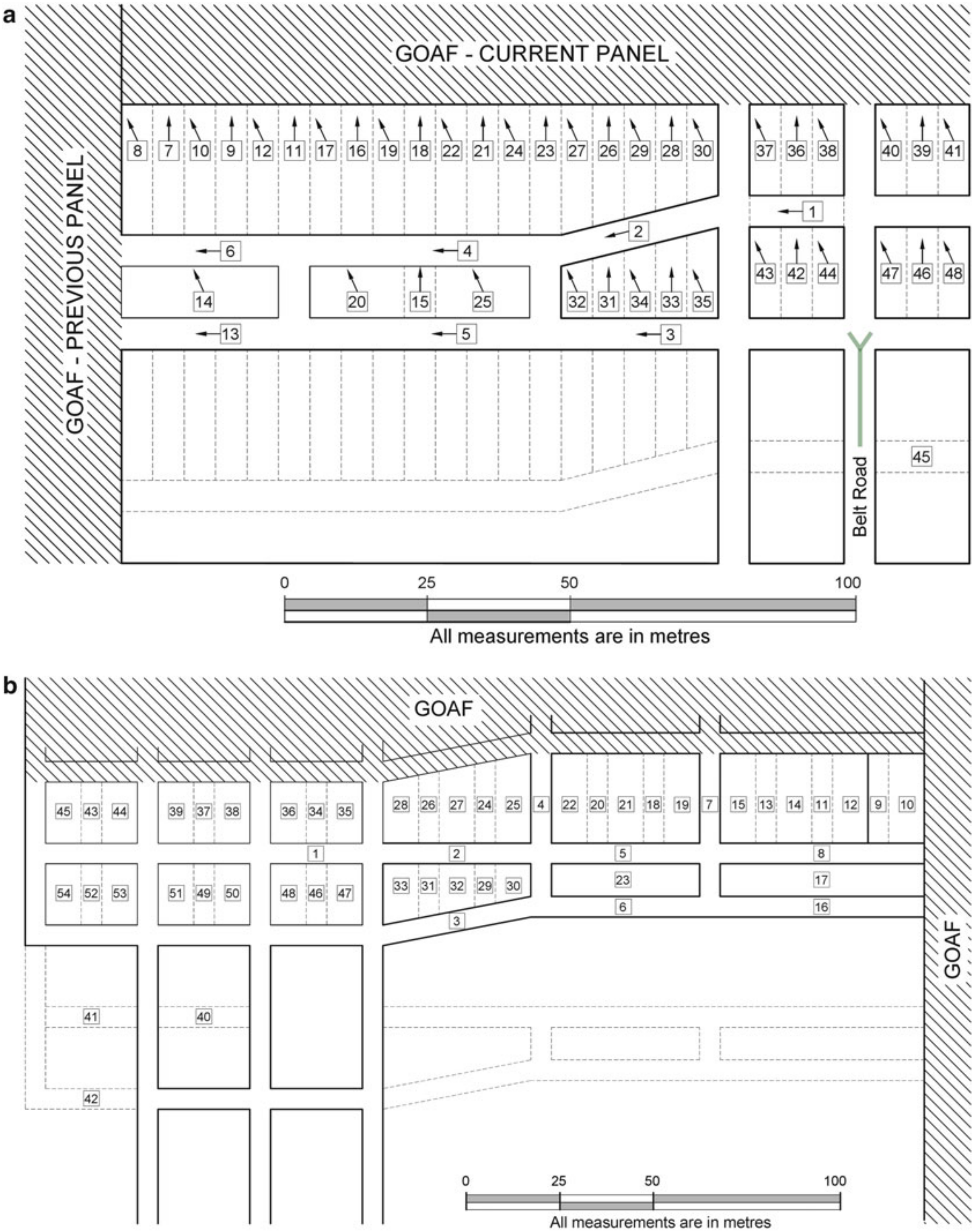

a
GOAF - CURRENT PANEL
GOAF - PREVIOUS PANEL
8 7 10 9 12 11 17 16 19 18 22 21 24 23 27 26 29 28 30
37 36 38
40 39 41
1
2
6
4
14
20 15 25
32 31 34 33 35
43 42 44
47 46 48
13
5
3
45
Belt Road
0 25 50 100
All measurements are in metres
b
GOAF
45 43 44
39 37 38
36 34 35
28 26 27 24 25
4
22 20 21 18 19
7
15 13 14 11 12
9 10
1
2
5
8
54 52 53
51 49 50
48 46 47
33 31 32 29 30
23
17
6
16
3
GOAF
41
40
42
0 25 50 100
All measurements are in metres

**Fig. 8.10** Old Ben and Modified Old Ben (Munmorah) methods of split and lift pillar extraction. (**a**) Old Ben method. (**b**) Modified Old Ben method

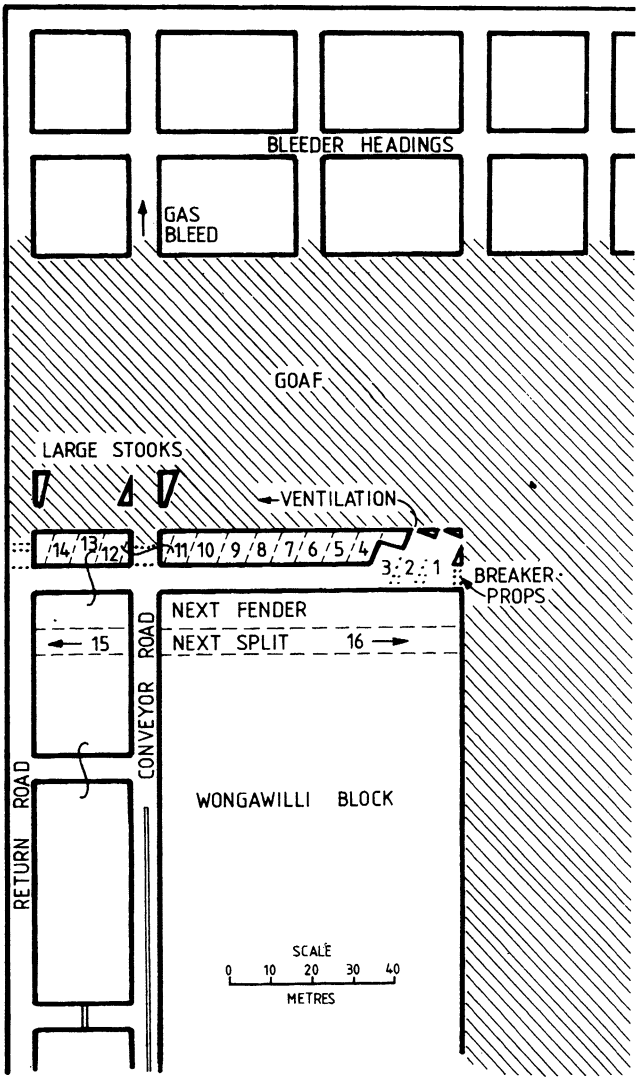

BLEEDER HEADINGS
GAS
BLEED
GOAF
LARGE STOOKS
VENTILATION
14
13
12
11
10
9
8
7
6
5
4
3
2
1
BREAKER
PROPS
NEXT FENDER
15
NEXT SPLIT
16
CONVEYOR ROAD
RETURN ROAD
WONGAWILLI BLOCK
SCALE
0
10
20
30
40
METRES

**Fig. 8.11** Wongawilli method of pillar extraction (After Sleeman 1993)

from the Old Ben method of pillar extraction to Wongawilli type layouts. However, these layouts have not proven particularly suitable for managing periodic weighting, causing a move towards partial pillar extraction systems.

In attempts to provide continuity to pillar extraction operations and to increase the rate of retreat, a range of continuous haulage systems have been developed. These systems have difficulty negotiating changes in direction greater than about 60°, thereby dictating the use of herring bone layouts that result in diamond shaped pillars. Examples of these types of layouts are shown in Fig. 8.15. Continuous haulage systems usually result in the panel being worked from the left and the right towards a centrally located belt road, rather than towards the side of the panel remote from the previously extracted panel.

The potential for the acute ends of diamond shaped pillars to spall, resulting in both an increase in roof span and an increased risk of rib fall injuries, takes on added significance in pillar extraction because the pillars are located in an abutment stress zone; some acute pillar corners are exposed to abutment stress on two

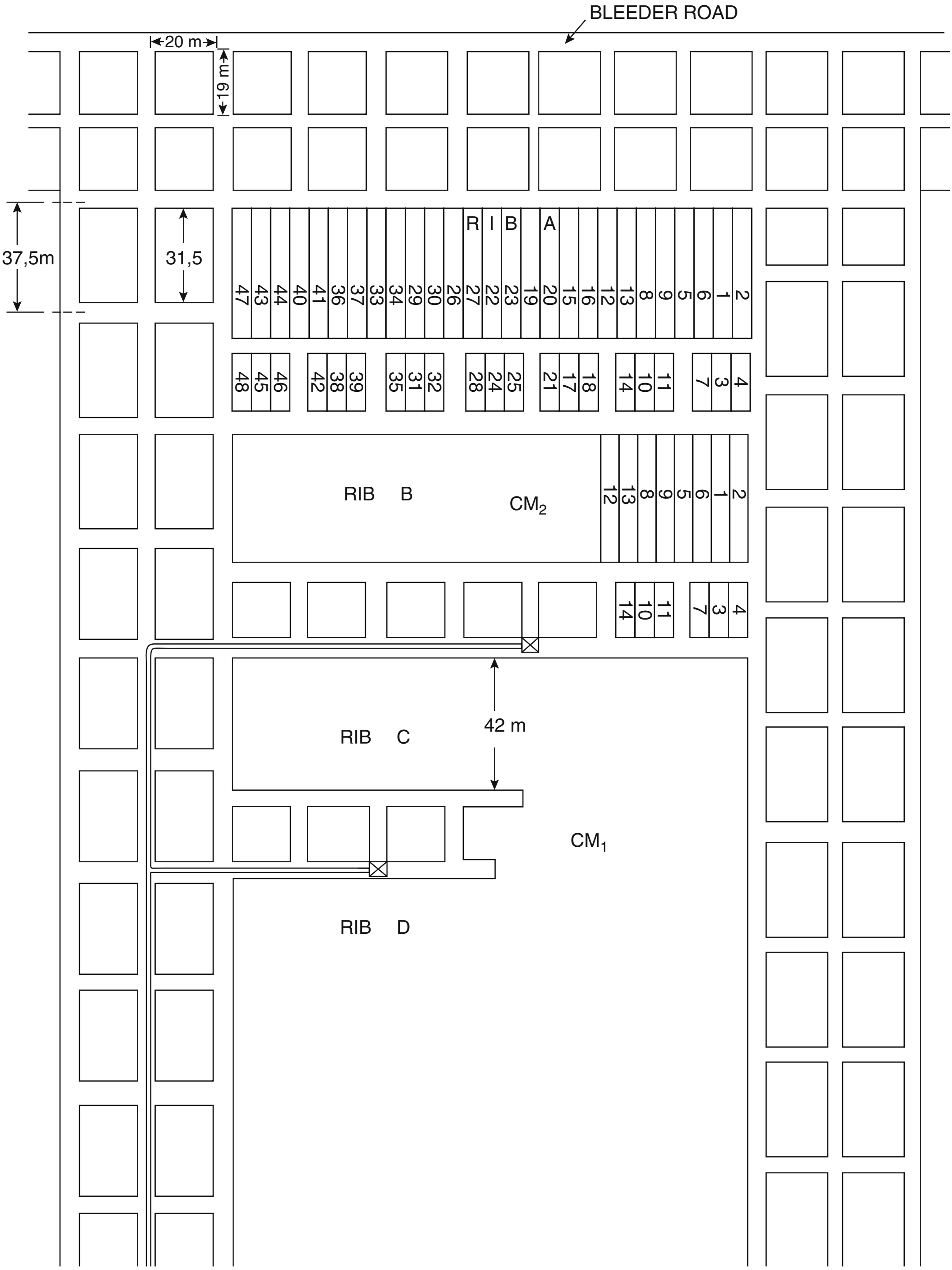

BLEEDER ROAD
20 m
19 m
37,5m
31,5
RIB A
47 43 44 40 41 36 37 33 34 29 30 26 27 22 23 19 20 15 16 12 13 8 9 5 6 1 2
48 45 46
42 38 39
35 31 32
28 24 25
21 17 18
14 10 11
7 3 4
RIB B
CM2
12 13 8 9 5 6 1 2
14 10 11
7 3 4
RIB C
42 m
CM1
RIB D

**Fig. 8.12** An example of the rib pillar extraction method with sequences designed to permit the use of two continuous miners (After Beukes 1990)

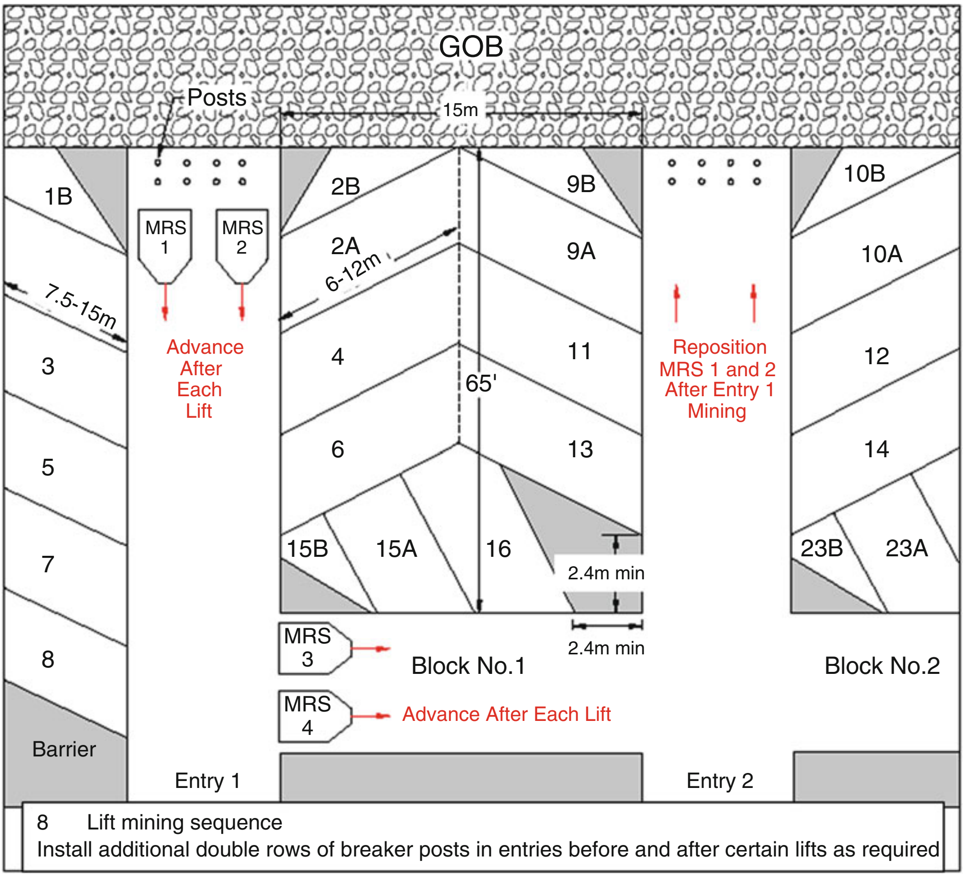

GOB
Posts
15m
1B
MRS 1
MRS 2
2B
2A
6-12m
9B
9A
10B
10A
7.5-15m
3
Advance After Each Lift
4
11
Reposition MRS 1 and 2 After Entry 1 Mining
12
65'
5
6
13
14
7
15B
15A
16
2.4m min
23B
23A
8
MRS 3
2.4m min
Block No.1
Block No.2
MRS 4
Advance After Each Lift
Barrier
Entry 1
Entry 2
8 Lift mining sequence
Install additional double rows of breaker posts in entries before and after certain lifts as required

**Fig. 8.13** An example of lifting left and right in a USA operation (Adapted from Marshall Miller and Associates 2006)

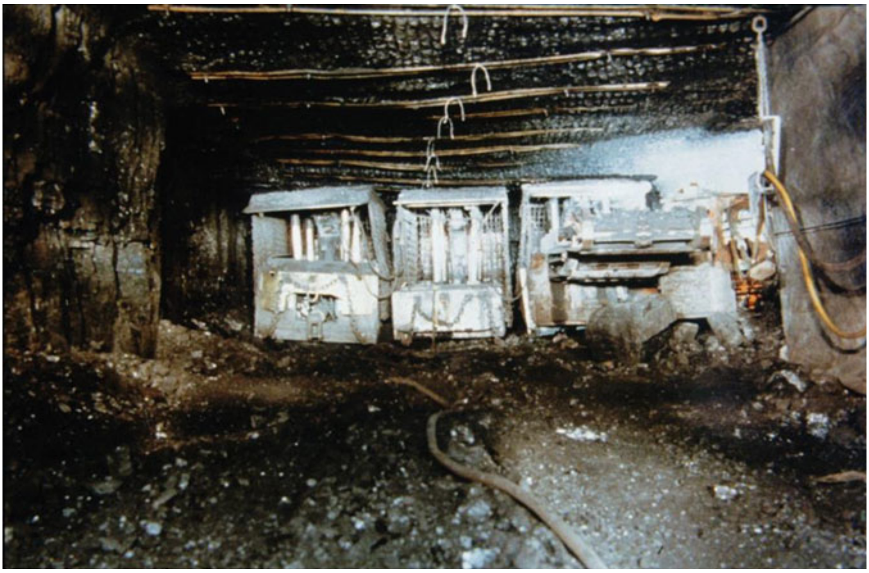


**Fig. 8.14** Lifting left and right using three MRS at an Australian colliery – right hand MRS partially obscured by the tail boom of the continuous miner

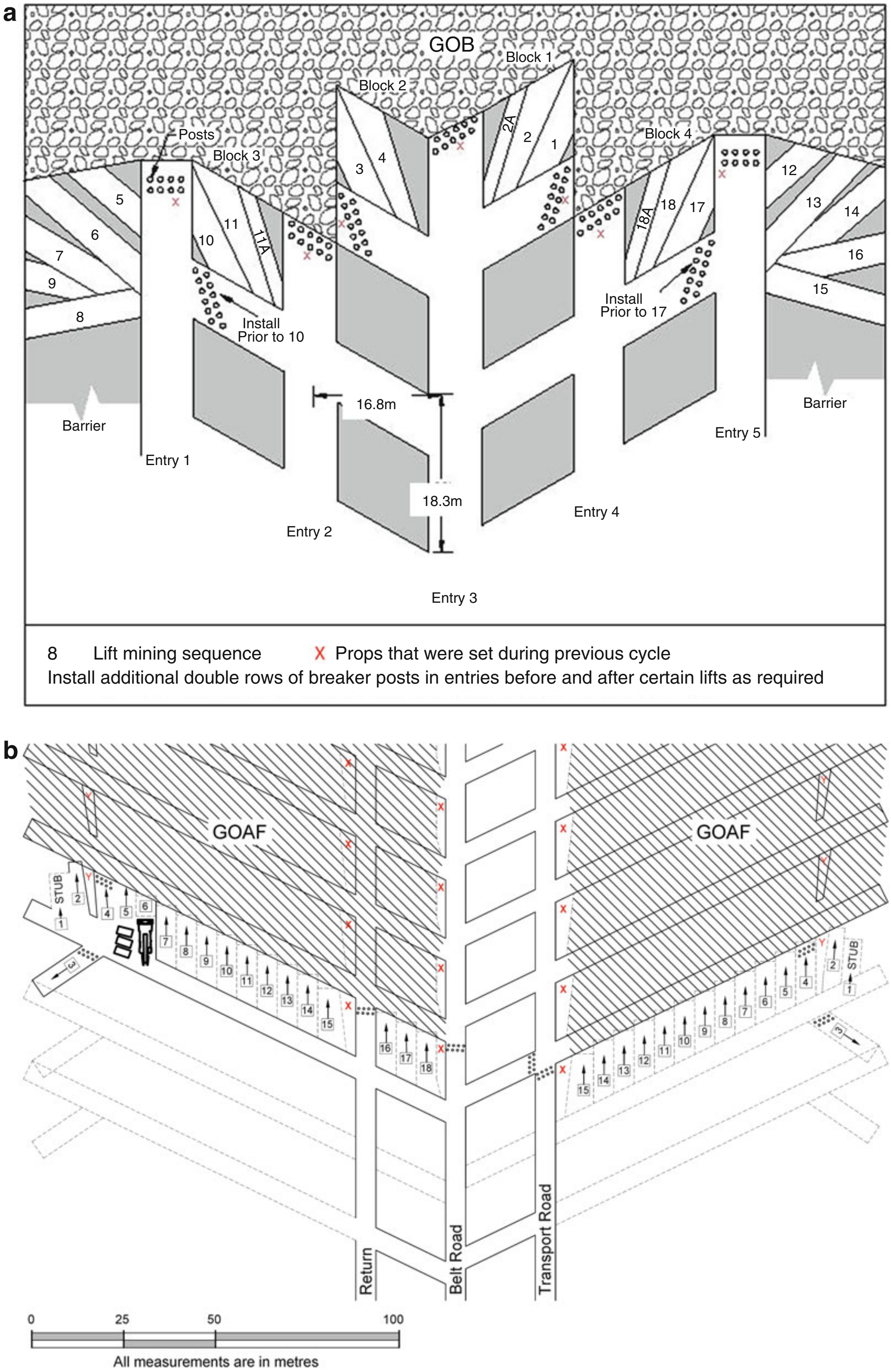

a
GOB
Block 1
Block 2
Block 3
Block 4
Posts
1
2
2A
3
4
5
6
7
8
9
10
11
11A
12
13
14
15
16
17
18
18A
Install
Prior to 10
Install
Prior to 17
16.8m
18.3m
Barrier
Barrier
Entry 1
Entry 2
Entry 3
Entry 4
Entry 5
8 Lift mining sequence
X Props that were set during previous cycle
Install additional double rows of breaker posts in entries before and after certain lifts as required
b
GOAF
GOAF
STUB
STUB
Return
Belt Road
Transport Road
0
25
50
100
All measurements are in metres

**Fig. 8.15** Two layouts for extracting pillars utilising a continuous haulage system. (**a**) An example of a USA layout and sequence plan when utilising continuous haulage (Adapted from Marshall Miller and Associates 2006). (**b**) An example of an Australian pillar extraction layout and sequence plan when utilising continuous haulage

sides since they project into the goaf; and the acute ends of fenders can be subjected to abutment stress from a previously extracted adjacent panel. In a continuous haulage operation, the last pillars extracted in each row usually project into the goaf at the centre of the panel (Fig. 8.15). Therefore, these pillars may be under higher abutment stress than the final pillars extracted on the virgin (block side) corner of layouts that utilise rubber tyre haulage, such as those shown in Figs. 8.10, 8.11, and 8.12.

Two further variations on pillar extraction involve extracting pillars on one side of a panel as it is being developed and mining top or bottom coal on the retreat out of a panel. The former is referred to as **pillar extraction on the advance,** with examples being shown in Fig. 8.16. This approach aims to improve ground control in high lateral stress environments by extracting some pillars as the panel is being developed so as to place the remaining pillars within a horizontal stress shadow at the time of their extraction. The concept has met with mixed success (Skybey 1984; Sleeman 1993).

#### 8.3.2.2 Shortwall Mining

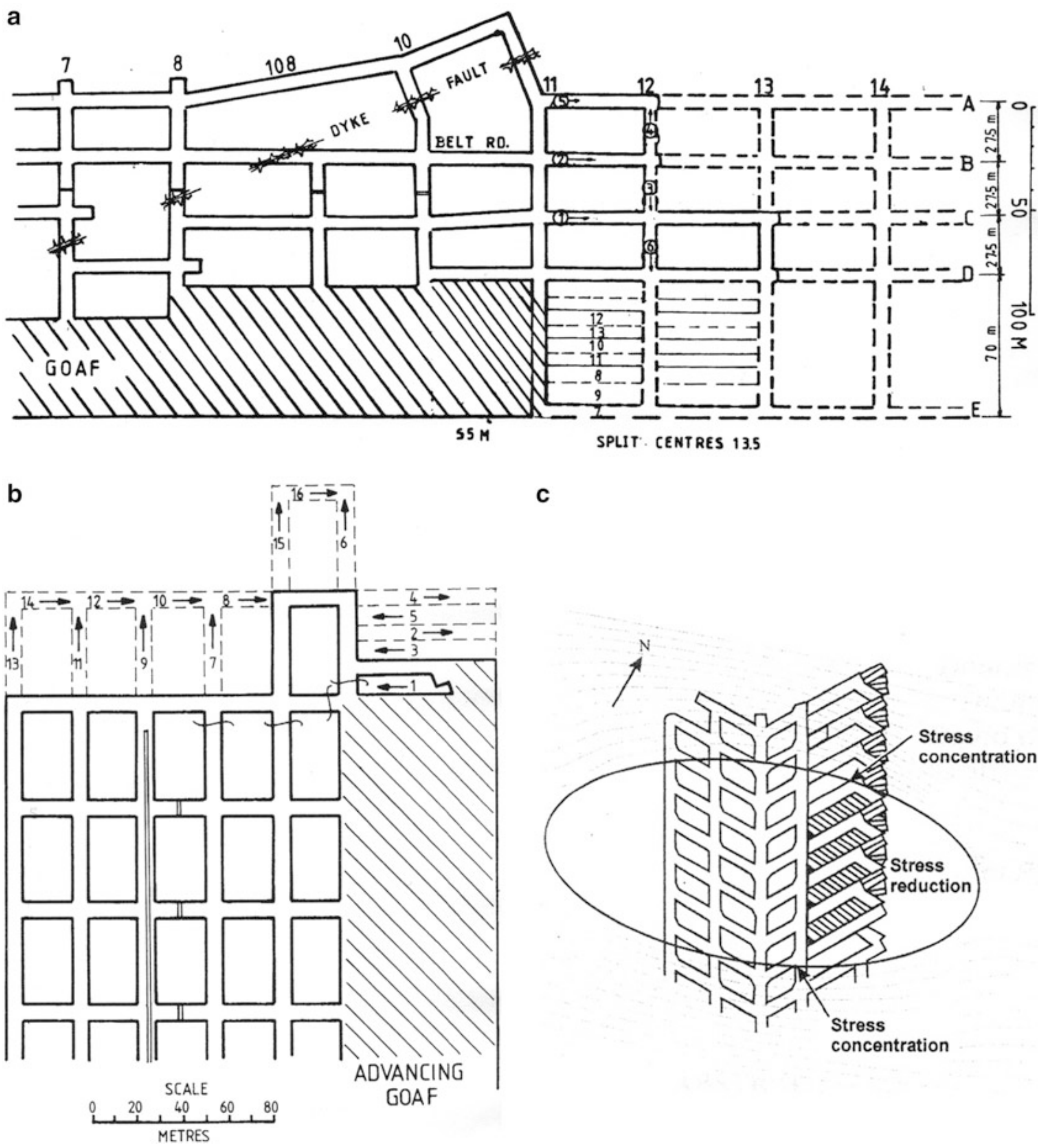
Shortwall mining was introduced into Australia in 1959 as an initiative to increase the extraction to development ratio (Monger 1994). It was based on a two heading longwall layout with a continuous miner and shuttle cars being used to take open ended lifts from the face and the chain pillars in panels up to 70 m wide. Face support initially comprised half round baulks and timber props that were replaced in 1968 with two leg Wild self-advancing longwall hydraulic supports. The method remained in use in Australia up to the late 1970s. At the time, it was also assessed for use in South Africa as a means of improving percentage extraction in massive and strong roof environments, where panel width had to be restricted in order to limit abutment stress associated with delayed caving or lack of caving of the superincumbent strata (Galvin et al. 1978; Wagner 2014). Subsequently, it has been periodically assessed for reintroduction in Australia but plans have not progressed. Major impediments with the method are a long canopy tip-to-face distance, a low support tip capacity, and coal clearance constraints.

#### 8.3.2.3 Hydraulic Mining

Hydraulic mining involves the use of high pressure water monitors to cut and transport coal from pillar extraction panels. It is suited to seams that are generally greater than 8 m thick and steeply dipping. Basically, a two heading development is driven at the base of the seam and to one side of the panel from which run-outs, referred to as ‘sublevels’, are mined to the rise at about 5°. A generic panel layout is shown in Fig. 8.17a. The minimum interval between sublevels is determined by pillar stability considerations, seam thickness and the maximum reach of the water monitors, which is typically limited to 30 m. Extraction is staggered between the sublevels and retreats by mining 15–20 m wide lifts (Fig. 8.17a). In the case of long narrow panels, such as shown in the New Zealand example in Fig. 8.17b, the panel development roadways may constitute the sublevels.

Duncan and Menzies (2007) and Smith (2011) describe the history of the method and the ways in which it has been adapted to thick seam operations in NZ. These include driving an additional roadway in the top of the seam on the opposite side of the panel to the main development, to function as a return airway. Features of the method that warrant consideration from a ground engineering perspective include:

- The roof of main development always comprises coal because these roadways are located towards the base of the thick seam.
- The durability of the floor strata in the sublevels and main development roadways, as coal is washed down these roadways. These strata are likely to comprise coal and some or all of the coal transport roadways may need to be concreted.
- The height of interpanel and barrier pillars abutting the extraction panels. These are often greater than 20 m in height and, as such, fall well outside empirical pillar design databases. Furthermore, while these pillars


a
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FAULT
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12
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27.5 m
27.5 m
27.5 m
70 m
0
50
100 M
GOAF
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SPLIT CENTRES 13.5
b
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12
10
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5
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3
1
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9
7
SCALE
0 20 40 60 80
METRES
ADVANCING GOAF
c
N
Stress concentration
Stress reduction
Stress concentration

**Fig. 8.16** Various layouts for conducting pillar extraction on the advance. (**a**) Layout associated with trials at Tahmoor Colliery, Australia (After Skybey 1984). (**b**) Layout based on the Wongawilli method (After Sleeman 1993). (**c**) Layout reported by Dolinar et al. (2000)

may give the impression in plan view of being large, they have low width-to-height ratios.

- The dip of the workings is conducive to ride between the roof and floor strata and to rib instability on the up-dip sides of pillars.
- Mine design needs to recognise that an open goaf can constitute a huge reservoir for gas and, therefore, goaf falls can have very serious implications for ventilation management and safety. Duncan and Menzies (2007) and

Panckhurst et al. (2012) describe controls for managing these situations.

#### 8.3.2.4 Longwall Mining

Galvin et al. (1991) and Bruins (1997) report on the use of a 73 m long conventional longwall face equipped with a single ended ranging arm shearer at New Denmark Colliery in South Africa to successfully extract 19 m square, 2 m high pillars in three bord and pillar sub-panels. The operations were conducted at a depth of 200 m under an immediate roof consisting of around 15 m of strong and massive sandstone, with the interpanel pillars comprising a single line of 19 m wide pillars. Figure 8.18 shows the stable ground conditions associated with longwall mining these sub-panels. The minimal support of the immediate roof of the headings and the upcoming longwall recovery roadway is noteworthy and reflects the stiff loading environment achieved by restricting the width-to-depth ratio of the sub-panels to around 0.4.

Bloemsma (1991) reported that this system provided the safest method of pillar extraction at the mine as all persons worked under supported roof and were not exposed to feather edging, which could over-run breaker lines by 8–9 m at that mine. Maximum surface subsidence of the order of only 100 mm confirms that the width-to-depth ratio of the sub-panels was subcritical and, hence, sub-panel pillars were protected to a degree from abutment stress.

### 8.3.3 Partial Extraction Methods

Duty of care obligations on employers not to expose employees to unacceptable risk, increased constraints on sub-surface and surface subsidence impacts, and the need to improve productivity have been catalysts in many operations moving from total pillar extraction systems to partial pillar extraction systems, particularly in Australia. Essentially, these systems are based on the same techniques as total extraction systems but restrict goafing, abutment stress

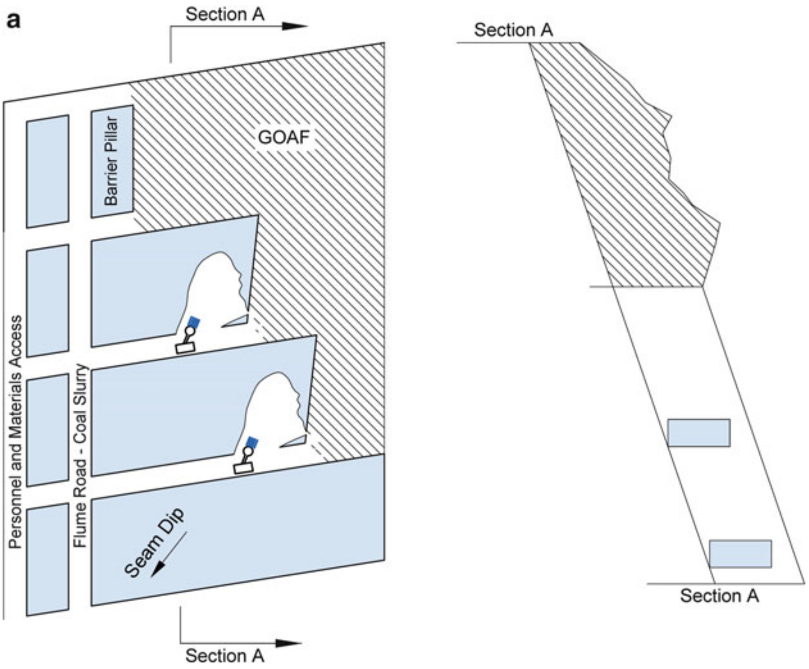
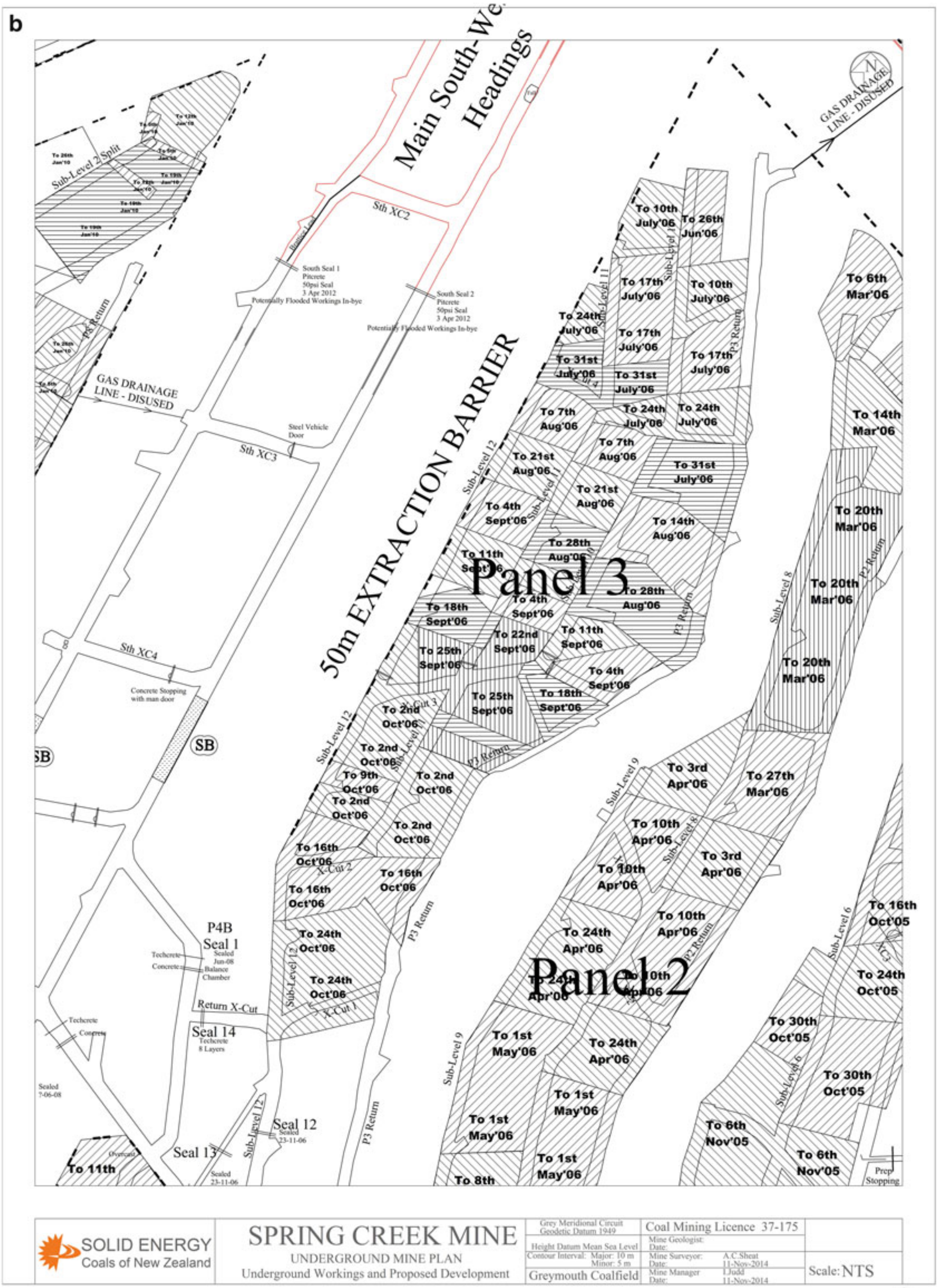


**Fig. 8.17** Mine layouts for hydraulic mining. (**a**) Conceptual layout. (**b**) Hydraulic mining between faults, Spring Creek, NZ


b
Main South-West Headings
Sth XC2
Sth XC3
Sth XC4
South Seal 1
Pitcrete
50psi Seal
3 Apr 2012
Potentially Flooded Workings In-bye
South Seal 2
Pitcrete
50psi Seal
3 Apr 2012
Potentially Flooded Workings In-bye
GAS DRAINAGE LINE - DISUSED
Steel Vehicle Door
Concrete Stopping with man door
50m EXTRACTION BARRIER
Panel 3
Panel 2
Sub-Level 2 Split
P3 Return
P2 Return
Sub-Level 6
Sub-Level 8
Sub-Level 9
Sub-Level 11
Sub-Level 12
X-Cut 1
X-Cut 2
X-Cut 3
SB
P4B
Seal 1
Sealed
Jun-08
Balance Chamber
Techcrete
Concrete
Return X-Cut
Seal 14
Techcrete
8 Layers
Seal 12
Sealed
23-11-06
Seal 13
Sealed
23-11-06
Sealed
7-06-08
Overcast
Prep Stopping
To 10th July'06
To 26th Jun'06
To 17th July'06
To 10th July'06
To 24th July'06
To 17th July'06
To 31st July'06
To 31st July'06
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To 24th Oct'05
To 30th Oct'05
To 30th Oct'05
To 6th Nov'05
To 6th Nov'05
To 11th
SOLID ENERGY
Coals of New Zealand
SPRING CREEK MINE
UNDERGROUND MINE PLAN
Underground Workings and Proposed Development
Grey Meridional Circuit
Geodetic Datum 1949
Height Datum Mean Sea Level
Contour Interval: Major: 10 m Minor: 5 m
Greymouth Coalfield
Coal Mining Licence 37-175
Mine Geologist:
Date:
Mine Surveyor: A.C.Sheat
Date: 11-Nov-2014
Mine Manager: I.Judd
Date: 11-Nov-2014
Scale: NTS

**Fig. 8.17** (continued)

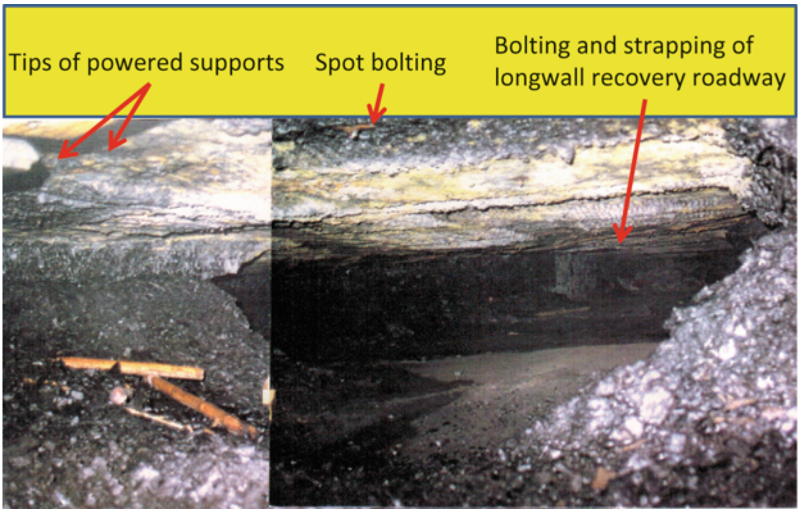


**Fig. 8.18** Physical conditions immediately ahead of a longwall face extracting standing pillars at New Denmark Colliery, South Africa (Modified after Galvin et al. 1991)

and subsidence of the overburden by extracting only certain rows of pillars or portions of pillars in each row. A matter of critical importance is the strength and stability of the pillars that remain unextracted in the goaf, since sudden failure of these can result in regional uncontrolled collapse with serious safety and environmental implications. Particular care has to be taken that unextracted and remnant pillars are stable in the long term. There is a range of partial pillar systems, examples of which are shown in Fig. 8.19.

Partial pillar extraction is not without its risks, one being that the absence of caving in some situations can lead to a false sense of security. Two of the five fatal falls of ground in Australian pillar extraction operations since 1991 (one a roof fall and the other a rib fall) occurred during pillar stripping in partial extraction panels.

At shallow to moderate depths, partial extraction methods can match, and sometimes exceed, coal resource recovery associated with total extraction. The pillar stripping method depicted in Fig. 8.19c for example, has a theoretical recovery rate of 67 % at a depth of 240 m, rising to 82 % at a depth of 80 m. However, care is required as the design of such layouts may fall outside the scope of conventional pillar design procedures.

When designing partial extraction layouts it is important not to focus only on the strength of the coal pillar element. Careful consideration must also be given to:

- The serviceability of the roadways in the abutment stress zone. Although the coal pillars themselves may be in an unfailed state in terms of maximum load carrying capacity, rib spall and convergence can still present serious direct and indirect risks to safety and productivity.
- Foundation failure. Floor heave, in particular, is often associated with pillar extraction, especially at depth.
- Excavation behaviour. Depending on the nature of the immediate roof and excavation span, partial extraction layouts can increase the risk of windblast. The potential for plug failure through to the surface needs to be evaluated when mining at shallow depth (less than 100 m but especially at depths less than 50 m – see Sect. 3.5).
- Pillar failure modes. Partial pillar extraction does not necessarily eliminate the risk of sudden pillar collapse and pressure bursts.
- Goaf edge control, including the merits of leaving stooks at intersections. The design of stable partial pillar extraction layouts can be deceptively complex, as reflected by failures in the Lake Macquarie region of NSW. Partial extraction layouts warrant

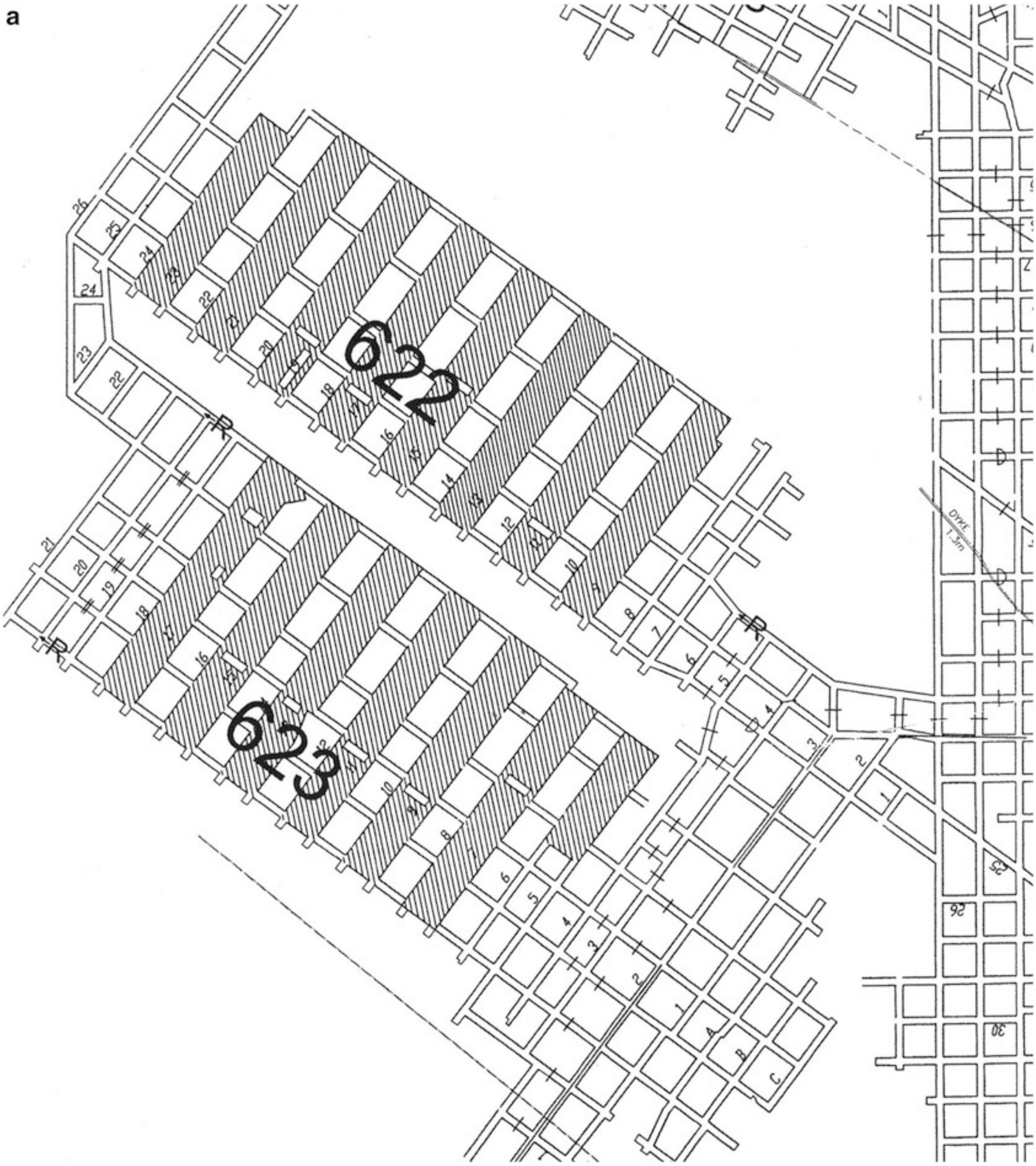
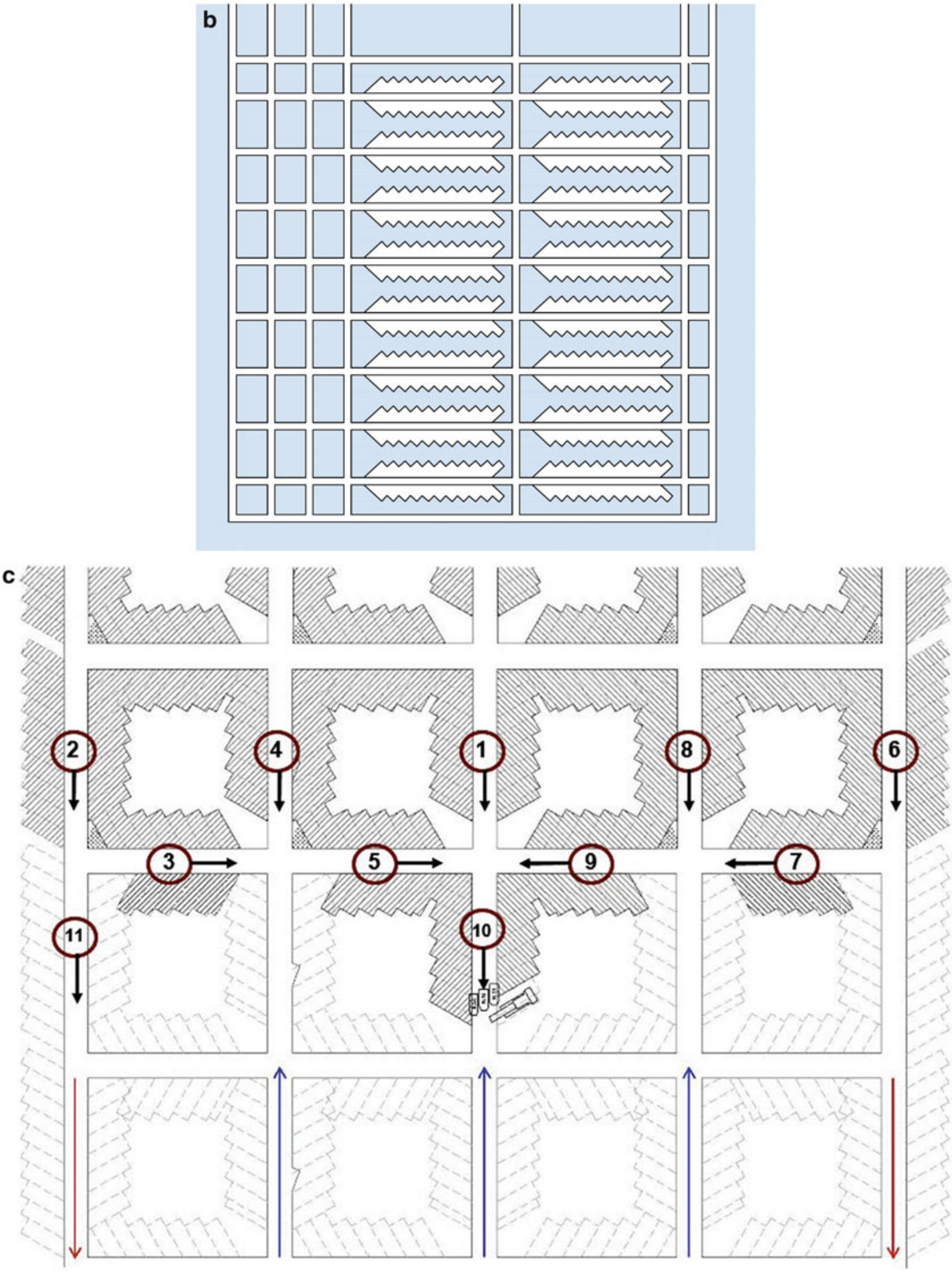


**Fig. 8.19** Examples of partial pillar extraction methods. (**a**) Panel and pillar mining based on 'take a row, leave a row', Myuna Colliery, Australia. (**b**) Concept of lifting left and right on two sides to form the final pillar size, based on a mine layout employed at Cooranbong Colliery, Australia (**c**) Lifting left and right on four sides to form the final pillar size, Tasman Mine, Australia (After Tyler and Sutherland 2011)


b
c
2
4
1
8
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11

**Fig. 8.19** (continued)

detailed geotechnical investigation prior to being implemented. This should include consideration of factors such as the impact of roof falls and flooding on the long term stability of the workings.

## 8.4 Ground Control Considerations

### 8.4.1 Introduction

Effective ground control in pillar extraction requires stability to be assessed at regional, panel and workplace levels. Some primary factors that impact on stability are interactive. Some operate at all three levels. These factors have been classified in this text according to the following criteria:

1. Mine: Factors that can have implications for the design of the overall mine layout rather than just one extraction panel.
2. Panel: Factors that can be varied on a panel-by-panel basis without affecting overall mine layout.
3. Workplace: Factors which can be varied on a pillar-by-pillar basis and impact primarily on workplaces in the vicinity of the extraction line.

### 8.4.2 Regional Stability

The principal factors impacting on regional stability are:

- superincumbent strata stiffness, which determines panel interaction, pillar loading and the manner in which pillars are likely to yield;
- existing workings in adjacent seams, which can be sources of concentrated loads that impact on both pillar stability and excavation stability;
- pillar system load capacity, including that of panel, interpanel, barrier and remnant pillars;
- pillar system functionality under abutment load, including periodic weighting load, with particular regard to rib and floor stability;
- spontaneous combustion propensity and management controls;
- gas content and ventilation controls;
- propensity for frictional ignition;
- major geological structure;
- cleat direction, dip and intensity; and
- caveability of the immediate roof strata, which can influence selection of mining span if there is potential for windblasts.

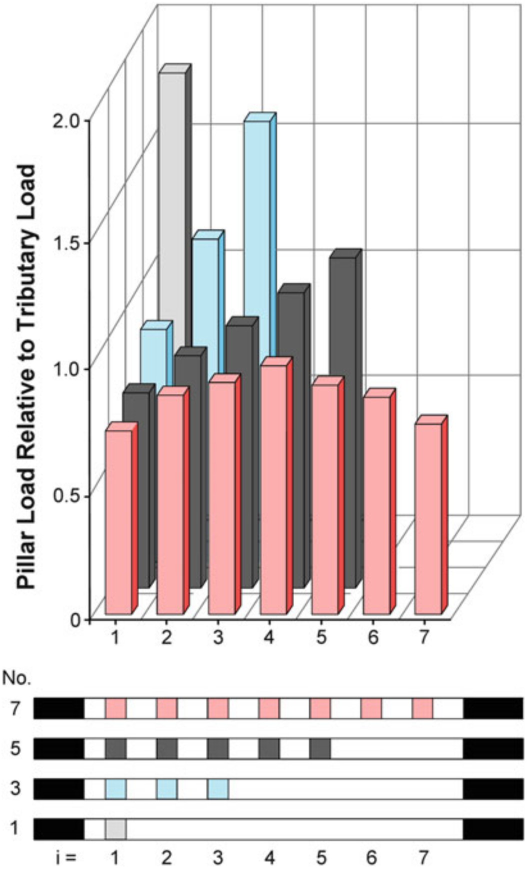


**Fig. 8.20** An example of change in abutment load associated with the superincumbent strata bridging over an extraction panel of increasing width (After Salamon 1992)

### 8.4.2.1 Load Distribution

Figure 8.20 illustrates how pillar load increases from that shown in Fig. 4.7 when pillars are progressively extracted prior to the onset of caving. The average pillar stress on the seventh row of pillars after six rows of pillars have been extracted has increased from about 0.7 times tributary area load to around 1.8 times tributary area load.

As the span of the totally extracted area increases, a point is reached where caving is initiated and the superincumbent strata then progressively subsides into the goaf as more pillars are extracted. Figure 8.21 shows simplistically the state of loading once the critical span, $W_c$, is reached and the superincumbent strata have fully subsided to produce maximum subsidence. The panel pillars at the extraction face line are subjected to the original tributary area load, $T_1$, plus a portion, A, of the load of the superincumbent strata that overhangs the goaf but is not supported by it, less an allowance for caving extending up to one side of the pillars ($T_1$–$T_2$). It should be noted that due to limitations associated with the concepts of caving angle and abutment angle (see Sect. 3.3.1), this model is restricted to the case of an isolated panel at shallow depth. It also makes no allowance for discontinuous subsidence associated with bridging superincumbent strata, which can have adverse consequences for pillar extraction operations because it increases abutment stress and the propensity for windblast and gas inrush.

A range of practical considerations constrains panel width in pillar extraction. Shuttle car cable reel capacities, for example, restrict wheeling distances to around 180 m, although this can be increased by back spooling (a practice often discouraged over extended distances due to the propensity for cable damage) and surging, whereby one shuttle car back-loads into another. The rate of pillar extraction retreat reduces with increase in panel width and this can have serious implications for not only strata control (because the strength of highly loaded rock can decrease over time) but also for the management of spontaneous combustion. Coals that are prone to spontaneous combustion have a characteristic incubation period and need to be smothered in the goaf within that time period. Ventilation method and volumes are other important practical considerations.

These types of considerations result in 200 m typically being about the upper limit for the width of a pillar extraction panel. This means that as depth increases beyond 200 m (and less for narrower panels), the superincumbent strata retains some stiffness (see Sect. 3.3.1). This can only be eliminated if the overall panel width is

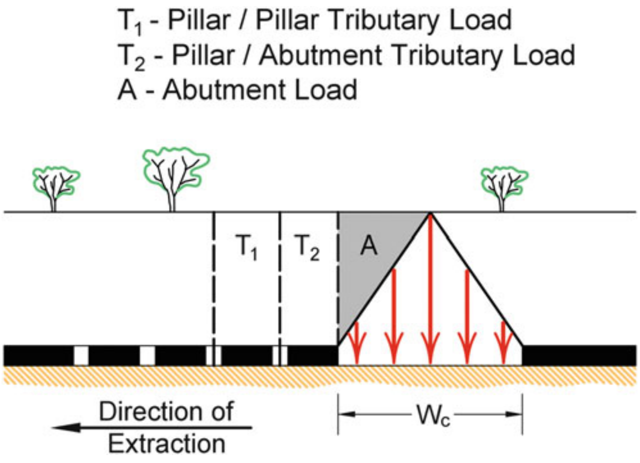

$T_1$ - Pillar / Pillar Tributary Load
$T_2$ - Pillar / Abutment Tributary Load
A - Abutment Load
$T_1$
$T_2$
A
Direction of Extraction
$W_c$

**Fig. 8.21** A simplistic representation of sources of pillar load around the perimeter of a pillar extraction goaf of critical width-to-depth ratio

increased by not leaving interpanel pillars when the next panel is extracted. However, at depths greater than about 200 m, high abutment stresses around the perimeter of the previous goaf and associated deterioration in ground conditions with the passage of time usually limit the opportunity to amalgamate goaves. Therefore, unless extremely wide interpanel pillars are formed, which is usually precluded by economic constraints, it becomes inevitable that adjacent pillar extraction panels increasingly interact as depth increases beyond about 200 m (see Sect. 5.2).

The load distribution about pillar extraction panels is complex and numerical modelling is required to aid in quantifying pillar load. This modelling needs to be three-dimensional to adequately evaluate interaction between multiple panels, especially at depth. Although it is limited to assessing behaviour around an isolated 150 m wide, 2.4 m high pillar extraction panel at a depth of 450 m, three-dimensional numerical modelling by Esterhuizen et al. (2010) based on the concept of a ground response curve provides further insight into the mechanics of strata behaviour. Figure 4.37 shows the effect of panel span on pillar load, with pillars of a width-to-height ratio of 6 predicted to already be in a critical state of stability at the time of formation. These pillars are predicted to fail as the pillar extraction line approaches, with equilibrium being restored at about 5.5 % vertical strain (Fig. 8.22).

The modelling predicts that pillars with a width-to-height ratio of 8 would be in a pre-peak stress state under development

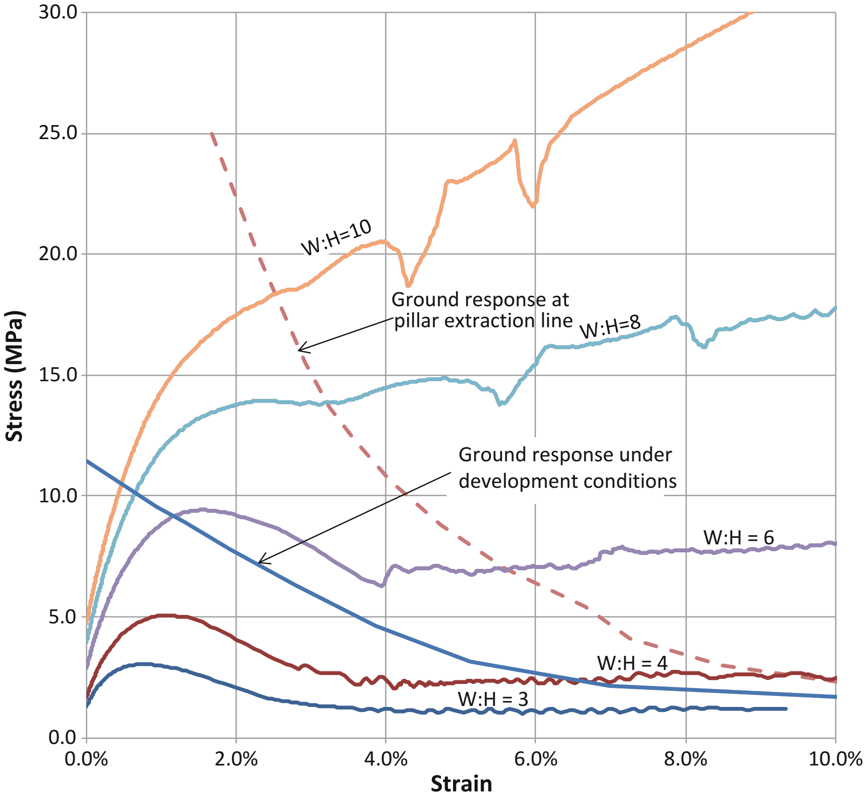


**Fig. 8.22** Pillar stress-strain curves and ground response curves at mid-span of a 2.4 m high, 150 m wide, 450 m deep panel under strong overburden, both immediately after development and during pillar extraction (After Esterhuizen et al. 2010)

conditions. As the extraction line approaches, the pillars are loaded beyond their initial peak and post-peak yielding occurs. At these relatively high stress values, the ground response is stiff and equilibrium is reached at a vertical strain value of 3.2 %. According to Esterhuizen et al. (2010), this level of strain is likely to be acceptable, since several case histories exist of successful pillar extraction under similar conditions.

#### 8.4.2.2 Pillar System Design

A starting point for pillar system design is the selection of mining height. In thinner seams, there may be no option but to extract the full seam thickness. However, thicker seams offer the potential to leave bottom coal as a barrier against soft or weak floor; to leave top coal to improve roof control; and to extract a select seam section for coal quality control purposes. Some pillar extraction techniques provide the option, at least on paper, to recover top or bottom coal on retreat out of the panel. However, the practice of mining additional coal from the roof on retreat is almost a thing of the past in continuous miner operations but taking bottom coal is common practice.

There are numerous variants of bottom coaling based on partial extraction of pillars and all involve ramping down into the floor and mining towards the goaf. Careful consideration needs to be given to the implications of the extra time that operations are exposed to the goaf edge; the stability and performance of support systems (especially timber props); the effects of reduced pillar stiffness; and the increased risk of rib spall. Importantly, pillar design must have regard to the final mining height in the life of a panel and to the likely state of the pillars at the time that the final extraction operations are being undertaken.

Four empirically based procedures for dimensioning pillars for subsequent extraction are those of Salamon and Oravecz (1976), Galvin (1994b), Galvin and Hebblewhite (1995) and Mark and Chase (1997). Salamon and Oravecz (1976) recommended that when using the Salamon and Munro formula (Eq. 4.23), the design safety factor of pillars which were to be subsequently extracted should be increased from 1.6 to a value of 1.8–2.0, depending on the extraction technique and the rate of extraction. In principle, increasing safety factor from 1.6 to 1.8 only improves the level of confidence that the strength of a pillar predicted by the Salamon and Munro formula will be greater than the average pillar stress based on tributary load theory. For a panel development safety factor of 1.6, the recommendation equates to increasing the design pillar load to 1.13–1.25 times the tributary area load. The subtlety with this approach, which is often not appreciated, is that it aims to maintain the level of confidence in the pillar design constant, at that probability associated with designing to a safety factor of 1.6.

Galvin (1994b) adopted the same philosophy in recommending that pillars at a goaf edge should be designed to carry as least twice the tributary area load. This was a ‘catch all’ recommendation based on experience that the additional pillar load carrying capacity catered for those situations where the development of caving was delayed, as for the case modelled in Fig. 8.20, or where panel width was sub-critical (see Sect. 3.3) and never resulted in the development of full caving. The recommendation reflects the important influence that the composition of the overburden strata has on abutment stress magnitude. When this strata is comprised of highly stratified, weak beds, abutment stresses are generally moderate to low whereas when the strata overburden contains strong and stiff beds in otherwise similar circumstances, abutment stresses tend to be relatively high.

Galvin and Hebblewhite (1995) combined the recommendations of Salamon and Oravecz (1976) and Galvin (1994b) in recommending that pillars in an extraction panel be designed on the basis of 1.3–1.5 times tributary area load, increasing to twice tributary area load for pillars which were to remain standing against a goaf edge for an extended period of time.

Implicit in the three preceding design approaches is that they apply to extracting standing pillars at depths of less than about 250 m. While the recommendations have generally proven successful, they should only be considered as 'rules of thumb'. Given the advances that have occurred in numerical modelling and risk management approaches to ground control, numerical simulation of final designs is advisable.

The NIOSH pillar extraction design methodology Analysis of Retreat Mining Pillar Stability (ARMPS) calculates stability factors which are based on estimates of pillar load and pillar load-bearing capacity during retreat mining (Mark and Chase 1997). The abutment angle concept in conjunction with the prescribed abutment load distribution profile given by Eq. 3.11 underpin the estimation of pillar load, while pillar strength is calculated using the Mark-Bieniawski formula (Eq. 4.30). Ground conditions in each case history used in the methodology have been categorised as either 'satisfactory' or 'unsatisfactory', with unsatisfactory conditions including controlled pillar failures (squeezes), sudden collapses of pillars, and pressure bursts. A case history was considered 'successful' only if no problems were reported and the mine plan showed that all pillars were recovered as planned.

A number of assumptions, approximations and rules of thumb are associated with the ARMPS methodology, particularly in regard to load magnitude and distribution. The methodology is based on a philosophy that the power of ARMPS is not derived from the accuracy of its calculations, but rather from the large database of retreat mining case histories that it has been calibrated against (Mark et al. 2011).

It was concluded from the 1997 model that when the ARMPS stability factor was greater than 1.5, pillar extraction designs were successful in 94 % of cases. The methodology was refined in 2002 with the goal of developing appropriate criteria for applying ARMPS to pillar design in deep cover situations (Chase et al. 2002). According to the ARMPS 2002 guidelines, a stability factor of 1.5 was satisfactory for pillar extraction cases where the depth of cover was less than about 200 m. Between depths of 200 and 400 m (~381 m if a strict conversion from imperial to metric is applied) there was a linearly decreasing trend in the required design stability factor. Beyond a depth of 400 m, a stability factor of 0.9 was recommended, dropping to 0.8 if the roof was strong. Two possible explanations were advanced by Chase et al. (2002) for this trend, namely:

- the actual strength of the large pillars at depth might be higher than predicted by the Mark-Bieniawski formula; and
- the pillar loads as predicted by ARMPS are higher than the actual pillar loads.

Heasley (2000) expressed the view that pillar loading was as important as pillar strength in pillar design. He questioned the accuracy of outcomes derived from the abutment angle/prescribed abutment load distribution profile approach when applied at depth, concluding from numerical analysis that ARMPS possibly over-predicted abutment load in deep cover cases. Tulu et al. (2010) noted that similar results were obtained by Colwell et al. (1999), who calculated significantly lower abutment angles for deep Australian longwall mines than the default value of 21° used in ARMPS and its related longwall chain pillar design methodologies Analysis of Longwall Pillar Stability (ALPS) and Analysis of Longwall Tailgate Serviceability (ALTS) (see Sect. 3.3.1).

As part of a study arising out of the Crandall Canyon Mine pillar failure, NIOSH re-evaluated the ARMPS program using an expanded case history database (NIOSH 2010). Empirical analysis, combined with numerical modelling, suggested that at depth, barrier pillars may be carrying more load than previously thought. Therefore, in updating ARMPS, the panel width-to-depth ratio, W/H, was added as an input parameter and the overburden load on the barrier pillars was increased by introducing a pressure arch load distribution over the panel pillars to address the inadequacies of tributary area loading at depth (Tulu et al. 2010). The formulation only applies to situations where the panel width-to-depth ratio, W/H, is less than one. Load is shifted to the barrier pillars either side of the panel by multiplying the panel pillar loads initially calculated on the basis of tributary area theory by a pressure arch factor (Mark et al. 2011).

NIOSH (2010) also made the following design recommendations to limit the risk of violent coal pressure bursts during retreat mining operations:

1. At depths exceeding 300 m (1,000 ft), pillar extraction mining should not be conducted without properly designed barrier pillars.
2. At depths exceeding 300 m (1,000 ft), pillar splitting should not be conducted on the pillar line.
3. At depths exceeding 600 m (2,000 ft), pillar extraction should not be conducted.

ARMPS is based only on evaluating the strength of the in-seam coal element of the pillar system. Stability may also be impacted by failure of other elements of this system and a number of other geotechnical factors not related to pillar failure. In particular, periodic weighting impacts manifested in severe rib spall, foundation failure, or featheredging are a common reason for abandoning pillars in the goaf. Hence, as advised by Chase et al. (2002) and others, ARMPS recommended stability factors should be considered as first approximation design guidelines that should be tempered with other site-specific variables deemed relevant on the basis of past experience and sound engineering judgement. Tulu et al. (2010) concluded from a range of simplified numerical analysis using the LaModel version of Salamon's laminated model, that the ARMPS methodology may benefit from further research into load shedding during pillar extraction. NIOSH (2010) concluded that there is still a considerable amount of variability in pillar design performance that ARMPS currently cannot explain.

The determination of load distributions and magnitudes about a pillar extraction panel becomes increasingly complex at depths greater than about 200 m. This is because any need to leave interpanel pillars most likely results in the stiffness of the superincumbent strata over the active extraction panel not being reduced to zero. An additional three or four panels may need to be extracted before stiffness reduces to zero above a panel, as illustrated in Figs. 3.16 and 3.19a. In the meantime, panel pillars are protected from full tributary load.

A pillar stability approach to pillar design based on the in-seam element of the coal pillar system does not guarantee safe and practical mining conditions at depth. As pillar width-to-height ratio increases and the pillars become squatter, stiffer and stronger, the likelihood that they constitute the weakest element in the pillar system reduces. Foundation bearing capacity failure, for example, may come into play. Irrespective of pillar size and average pillar strength, a point is reached where the strength of the unconfined pillar edge is exceeded and rib failure is initiated.

Avoidance of bearing capacity failure is one reason why Salamon and Wagner (1979) recommended a minimum pillar width-to-height ratio of 15 for interpanel (stabilising) pillars used in South African deep gold mines. Delaying the splitting of pillars in the extraction line and maximising the speed of pillar extraction are also critical for controlling foundation failure during pillar extraction.

At depths less than 150–200 m, the determination of panel pillar load is more straightforward because panel width is more likely to be critical to supercritical. In the absence of strong stiff beds in the superincumbent strata, it may be reasonable in these situations to approximate abutment load using the abutment angle concept in conjunction with a prescribed abutment load distribution profile. This is because the height of caving and fracturing is a considerable proportion of the overburden depth and, as illustrated in Figs. 3.14 and 3.16, panels largely behave independently of each other when separated by interpanel pillars. In all other cases, a more detailed geotechnical study is recommended.

#### 8.4.2.3 Panel Span Considerations

In Sect. 3.3.3 it was noted that, ideally, the span of an extraction panel should be sufficiently wide so as to induce full caving and subsidence of the overburden very soon after the commencement of secondary extraction in order to achieve some relief from abutment stress, or else sufficiently narrow so as to restrict abutment stress. However, this is not always practically achievable, particularly at depth. It was also noted that an extreme situation can arise when secondary extraction takes place at relatively shallow depth (typically less than 200 m) in a panel with a span that is only marginally less than that required to induce full caving. In these cases, the face extraction line can be subjected to high abutment stress throughout the life of the panel; ground control is very susceptible to small changes in geology; and localised caving may occur on an irregular basis. Strata behaviour is characterised as unpredictable and inconsistent, with even minor changes in lithology being sufficient to trigger an unexpected fall of ground. When the panel span is supercritical, the presence of strong and stiff strata in the overburden may still cause caving to be delayed, resulting in periodic face weighting and the risk of windblast.

These circumstances give rise to particularly hazardous situations in pillar extraction for two reasons. Firstly, the face line does not comprise a solid coal abutment but includes headings, cut-throughs and intersections which all present an elevated risk of instability. Secondly, unlike in longwall mining, there is no physical barrier between face operations and the roof or the goaf at these locations. Sustained high abutment stress and periodic weighting both contribute to having to leave large portions of pillars unextracted and larger than designed stooks, which then serve to aggravate the situation. Ultimately, the extraction face may have to be abandoned and re-established two or three rows of pillars further outbye, as illustrated in practice in Figs. 8.23, 8.24 and A9.1.

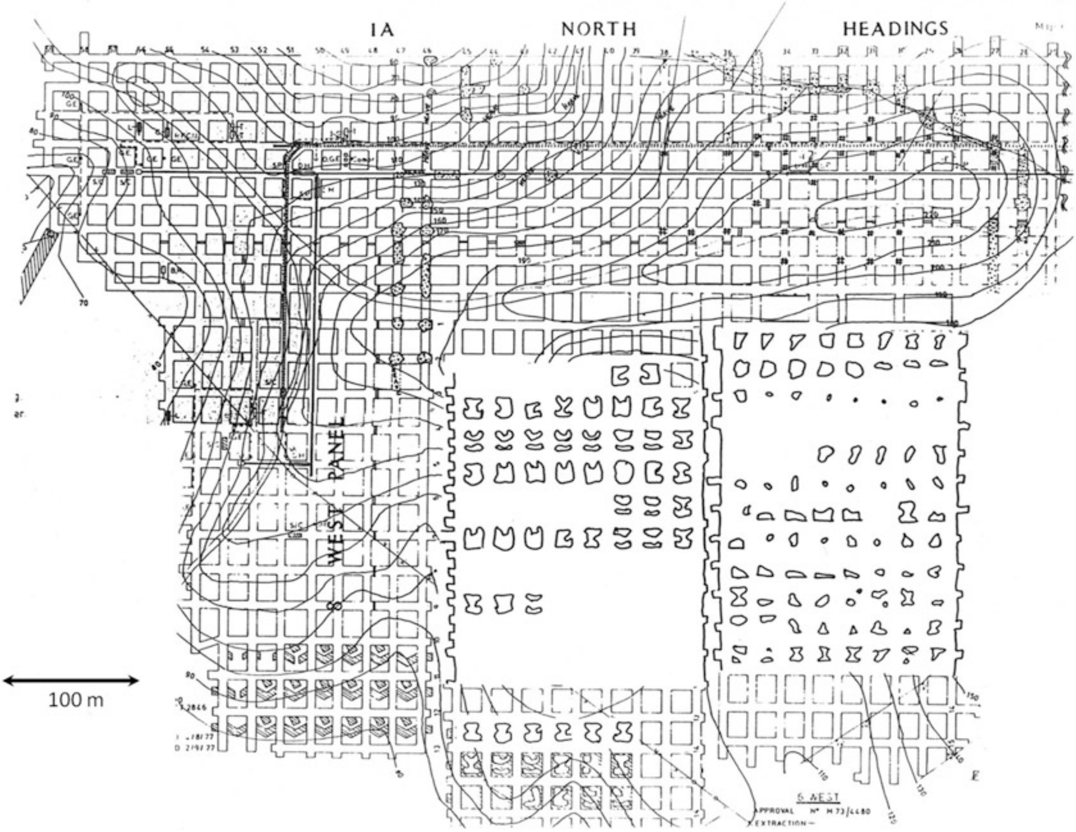


**Fig. 8.23** An example of the impact of incomplete caving (subsidence) and periodic weighting on the total extraction of standing coal pillars

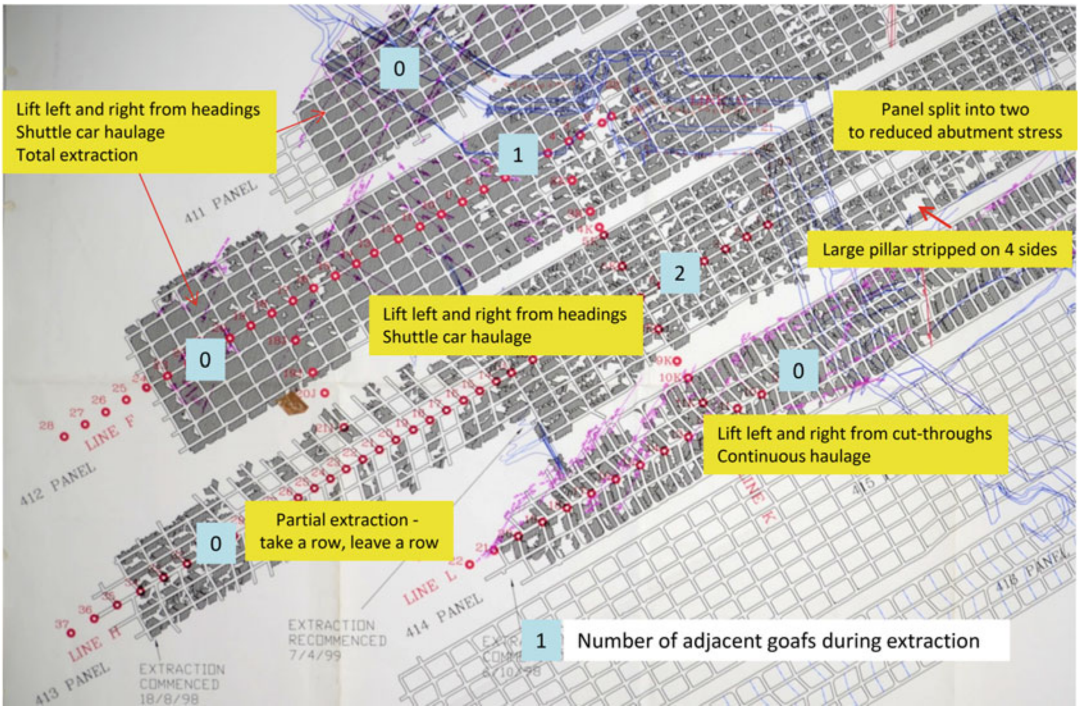

Lift left and right from headings
Shuttle car haulage
Total extraction
0
1
Panel split into two
to reduced abutment stress
Large pillar stripped on 4 sides
2
Lift left and right from headings
Shuttle car haulage
0
0
Lift left and right from cut-throughs
Continuous haulage
Partial extraction -
take a row, leave a row
0
1
Number of adjacent goafs during extraction
411 PANEL
412 PANEL
413 PANEL
414 PANEL
415
416 PANEL
LINE F
LINE H
LINE L
LINE K
EXTRACTION
RECOMMENCED
7/4/99
EXTRACTION
COMMENCED
18/8/98

**Fig. 8.24** An example of regional pillar extraction design principles in practice

At shallow depth, the option may be available to manage high abutment stress on the extraction line by initially restricting panel width and not leaving interpanel pillars. This is in order to form an excavation of overall supercritical span during extraction of the second or third panel. Other options include narrowing a panel once the problem becomes apparent or leaving additional pillars towards the centre of the panel to reduce effective span. Care is required with the latter approach to avoid creating a situation where a sudden pillar collapse occurs in the goaf, with implications including inrush of noxious and flammable gas, windblast and goaf over-run into the workplace.

The hazards associated with incomplete caving and periodic weighting in pillar extraction and the exposure of operators to falls of ground are primary reasons for the large range of pillar extraction methods, panel spans, and manners and sequences for extracting pillars. These have evolved largely from operational experience through trial and error, as reflected in the regional place names of some techniques such as Wongawilli, Old Ben and Munmorah, and contribute to why pillar extraction is considered by many to be an art as much as a science. The options available for designing panel span are reviewed in Sect. 3.3.3.

#### 8.4.2.4 Spontaneous Combustion, Ventilation and Frictional Ignition

Propensity for spontaneous combustion is an important consideration when selecting panel span because pillar extraction operations must retreat at a rate sufficient to cause coal left in the goaf to be smothered within its incubation period. A balance needs to be struck between the panel being too wide to achieve the required rate of retreat and too narrow to result in adequate caving and compaction to prevent the ingress of air to the goaf.

The gas content of the coal and the ventilation layout of the mine are important considerations in their own right and integral to spontaneous combustion management. Gas pre-drainage is a control for reducing risk associated with flammable and noxious seam gases. However, it does not eliminate the risk of explosion. In the case of coals not prone to spontaneous combustion, it is standard and good mining practice to maintain a differential ventilation pressure across a goaf in order to bleed gas make from it. This requires that either main development roadways flank the starting and finishing ends of each panel or else select roadways, referred to as **bleeder roadways**, are maintained in an open state around the perimeter of pillar extraction panels. If goaves are not bled by means of bleeder roadways or surface to seam boreholes, elevated risk is associated with goaf falls causing an inrush of flammable and noxious gas at the workplace.

In the extreme case of a windblast, flammable gas may be forced into outbye areas of the mine where electrical equipment is not required to be intrinsically safe or explosion protected. Propensity for windblast is an important consideration when determining the number of headings comprising a pillar extraction panel and how the panel is connected to surrounding workings. This matter is discussed in more detail in Sect. 11.1.

A conundrum arises when a seam is both gassy and prone to spontaneous combustion as bleeding air across a goaf promotes spontaneous combustion. The bleed can provide sufficient oxygen to support oxidation of broken coal but insufficient air flow to remove heat at a rate necessary to prevent the oxidation process accelerating into open combustion. The requirements of government regulators can be an added complication since some jurisdictions have a preference for all goaves to be ventilated while others are strongly opposed to ventilating goaves that are prone to spontaneous combustion. Some operations address this issue by piping nitrogen into goaves to displace flammable and explosives gases to bleeder airways and goaf drainage boreholes while, at the same time, controlling the ingress of oxygen into the goaf.

Spontaneous combustion can present a higher risk in the case of partial pillar extraction panels. This is because, firstly, falls of roof may be shallow or not occur at all and so broken coal in the goaf is not effectively smothered. Secondly, the large open voids invariably result in very low ventilation velocities through goaf areas, thus promoting oxidation of coal but not adequately removing heat. Blind working places present an elevated risk.

At the time, the most probable source of ignition for the Moura No. 4 explosion in Australia in 1986, which was associated with a goaf fall in partial extraction workings, was attributed to a flame safety lamp (Lynn et al. 1987). However, frictional ignition involving caving sandstone could not be ruled out and there remains a strong view amongst some that this was the ignition source. The Moura No. 2 disaster in Australia in 1994, described by Roxborough (1997), provides a good case study of the ventilation and spontaneous combustion issues that need to be considered when designing pillar extraction workings.

The précis of these two events and the Endeavour Colliery explosion in Australia in 1995 provided in Appendix 9 illustrate the preceding principles in practice. In many instances, spontaneous combustion, ventilation and frictional ignition involving rock on rock contact can be the most important factors in deciding if pillar extraction can be undertaken safely and, if so, in determining the required mining layout, mining method and mining dimensions. Further information on these aspects is contained in Humphreys and Richmond (1986), Ward et al. (1990), Cliff et al. (1996) and MDG-1006 (2011).

#### 8.4.2.5 Geological Structures

Geological structures can impact on pillar extraction design at a regional level by determining the orientation, width, and start and finish points of a panel. Safety and speed of development considerations may dictate that panels are laid out between major geological structures. If geological structures are present in a panel, it is judicious and often imperative that pillars are not extracted in their vicinity. This can have implications for re-establishing caving. On the other hand, if conditions do not present an unacceptable risk to safety, geological discontinuities can be used to advantage by commencing a panel against them to promote the early onset of full caving.

#### 8.4.2.6 Cleats and Joints

Rib fall is a well established and serious risk in pillar extraction. Chase et al. (2002) reported that at higher mining heights, miners almost always indicate that one needs to pay more attention to the ribs than to the roof. Statistics compiled by the USA Mine Safety and Health Administration (MSHA) indicate that, on average, deep cover pillar retreat miners are three times more likely to be injured by rib falls than others engaged in bord and pillar mining (NIOSH 2010).

Cleat and joint direction, dip and intensity have a major influence on rib stability and, therefore, need to be taken into account in panel design. This can be particularly challenging, even if there are no other factors that conflict with optimising panel orientation to minimise the impact of cleating and jointing, because pillar extraction methods involve mining in at least three to four different directions (heading, cut-through, lift left, lift right). This makes it difficult to avoid mining sub-parallel to these features in at least one direction. Furthermore, rib support may be restricted to cuttable type products that have lower support capacities than steel based products, with full steel meshing of the ribs not feasible.

In all instances, it is advisable to keep operators well back from the extraction line and away from ribs. If roof conditions and statutory requirements permit, one control measure is to drive wider roadways. This does not decrease the probability of a rib fall but is intended to reduce its consequences by allowing operators to stay further away from the ribs. If it is unavoidable that cleating and jointing is orientated sub-parallel to one mining direction, then it is usually best that the lifts correspond with this direction. Accidental tripping of emergency stop buttons by falling ribs and an appropriate safe working procedure for resetting the buttons take on added importance in these circumstances.

#### 8.4.2.7 Multiple Seam Workings

From a regional perspective, the presence of workings in an adjacent seam may predetermine panel dimensions and/or the location of headings and cut-throughs when designing pillar extraction workings. The influence of multiseam workings has been discussed in Sect. 5.3 but takes on added significance if these are pillar extraction workings because of the likelihood that pillars were not fully extracted or always extracted in the planned manner and sequence. Elevated stresses associated with remnant pillars and panel abutments in one seam can also have implications for pressure burst in workings in adjacent seams. Hence, it is important to keep accurate pillar extraction records if future workings are a possibility in neighbouring seams and to be particularly alert to the implications of inaccurate extraction records for pillar extraction workings in adjacent seams.

#### 8.4.2.8 Principles in Practice

The pillar extraction workings illustrated in Fig. 8.24 are an example of the range of variations on pillar extraction that exist at some mines and of the application of some of the preceding principles regarding regional stability. Points of particular note are:

- the coal had a moderate to high propensity for spontaneous combustion and so the mine did not employ bleeder ventilation;
- mining methods involve both square and diamond shaped pillars (reflecting different coal haulage systems), partial and total extraction methods, and pillars being stripped from both headings and cut-throughs and on up to four sides;
- there is a clear correlation between the number of adjacent panels (0, 1 or 2) and pillar recovery, reflecting the impact of abutment stress;

- in an initiative aimed at reducing abutment stress impacts on rib stability in the most adverse case of pre-existing flanking pillar extraction panels, 413 panel was divided into two narrow panels mid-way through extraction by leaving a central pillar unextracted in each row.

### 8.4.3 Panel Stability

Assessment of ground control on a panel basis should include consideration of:

- local geological structure;
- the type and effectiveness of primary support systems;
- manner and sequence of extraction;
- fender width;
- intersections and stooks;
- in all instances, but particularly in the case of standing pillars, design versus actual dimensions and off-centre drivages; and
- secondary support systems and strategies.

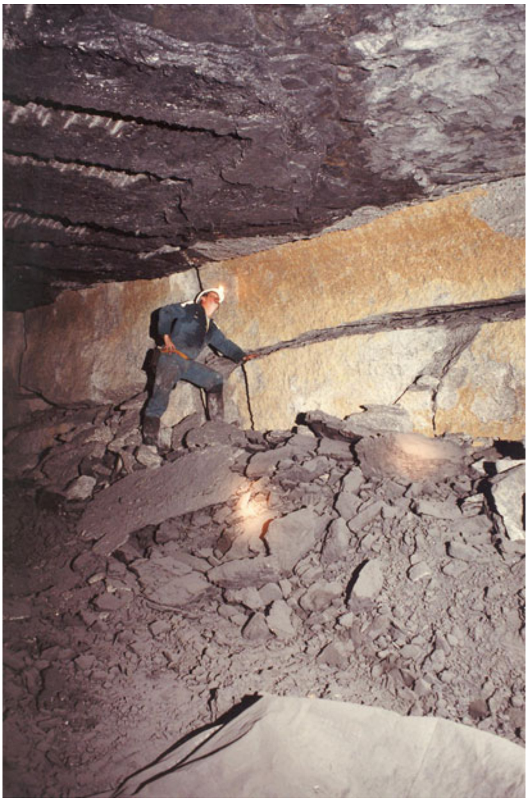


**Fig. 8.25** A goaf fall along a pre-existing open joint plane or 'water crack'

#### 8.4.3.1 Local Geological Structure

Local geological structure is structure that is not sufficiently well developed to influence the position and dimensions of a panel but which can have a major influence on ground control within the panel. This could be reflected in a local deterioration in roof, rib or floor conditions and unpredictable or uncontrolled caving behaviour. Hazardous conditions can be associated with an extraction line that is orientated near parallel to geological structure, as this can provide the opportunity for the cantilevered roof strata to shear and fall outbye of the working face. Joint direction and appearance can be good indicators of the state of stress. Joint planes orientated normal to the major horizontal stress direction may be tightly closed and difficult to detect outbye of the working face. Wet, open joints, sometimes referred to as **water cracks**, are usually orientated normal to the direction of a tensile horizontal stress and, therefore, risk is elevated when they align with a goaf edge, such as for the case shown in Fig. 8.25.

The appearance of stone dust can assist in the detection of discontinuities if they are damp or active. However, stone dust can also disguise features. It is advisable, therefore, to carefully inspect for geological structures prior to the application of stone dust; to highlight any structures with paint traces; and not to stone dust over these traces. This aids in making their location and trend readily apparent to all persons on all shifts in the workplace.

Usually, the need to leave coal around a geological structure can only be determined on a site-specific basis. There may be no option but to do so if roof, rib or floor conditions associated with the structure impede safety or speed of extraction. Otherwise, the orientation of the structure relative to the extraction line is a primary determinant.

A number of instabilities in partial extraction workings have been associated with compensating for coal left around a geological structure in permissible mining segments by extracting coal from neighbouring segments that were designed to be left unmined. Whist such modifications to the mine plan are not out of the question, they have the potential to impact adversely on stability. Therefore, they should be subjected to a formal change management process based on risk assessment and involving competent ground engineering personnel. In all circumstances, not just pillar extraction, the support capacity of structurally disturbed ground should be carefully assessed.

#### 8.4.3.2 Existing Workings and Support Systems

Existing workings may comprise main panel development, panel pillars purposely designed to be extracted, or standing pillars that were not designed with subsequent pillar extraction in mind. In all cases, but particularly in the case of old standing pillars, it is important to undertake a survey of actual dimensions and geological and geotechnical conditions prior to the commencement of secondary extraction.

Pillar dimensions in general are an important consideration in determining the manner and sequence of extraction. Ideally, pillar dimensions should be a multiple of lift widths, allowing for any fenders or stooks that may need to be left. Over-width and off-centre drivages can have serious implications for safely implementing these plans as they can lead to unstable situations due to fenders and stooks being either undersized or oversized. Undersized fenders can result in loss of protection from caving and the goaf over-running into the workplace. Oversized fenders may not be able to be fully extracted and so impede caving. They can inadvertently or otherwise cause operators to venture beyond supported roof in an attempt to extract them. Once the manner and sequence for extracting each pillar has been determined, it is good practice to mark (with paint) the location and sequence number of the lifts on the sides of the pillars.

The pre-mining geotechnical survey should include an assessment of the condition of the primary support system and any need to supplement it with secondary support. The manner and sequence of extracting pillars at specific locations may need to be modified to take account of existing poor ground conditions or the state of the existing support systems. This could include, for example, restricting access to some cut-throughs and headings in order to limit the extent of areas requiring re-support or supplementary support.

#### 8.4.3.3 Manner and Sequence of Extraction

The manner and sequence of pillar extraction is critical for maintaining a safe and productive workplace. Two primary objectives should be to, firstly, provide workplaces for personnel that are located between solid coal abutments, under the protection of supported ground, and removed from the immediate vicinity of coal ribs, pinch points and goaf edges; and, secondly, to maintain a relatively straight face line to minimise the opportunity for high stress concentrations and variable ground behaviour.

There is considerable potential to vary the manner of extracting coal within a panel while still achieving regional design outcomes, as evident by the pillar extraction methods reviewed in Sect. 8.3.2 and shown in Fig. 8.24. Once the basic manner for extracting coal has been decided, the sequence for extracting pillars and lifts within individual pillars has to be determined. This may be fixed for the entire panel or vary from pillar to pillar if the pillars are of non-uniform or irregular shape. Ideally, the sequence of extraction should be simple and repetitive to minimise the potential for confusion in executing it. The basic principles to be followed include:

- avoid simultaneously weakening multiple pillars on the extraction line by not adopting practices such as pre-splitting and place changing;

- avoid arrow head layouts whereby a pillar may end up jutting out into the goaf and be surrounded by goaf on three sides;
- maintain a relatively straight extraction line (a step in the extraction line is unavoidable when extracting standing pillars);
- consider the physical location of operators and the practicality of a lifting sequence being executed without operators being exposed to rib spall, unsupported roof and pinch points;
- do not lift off a pillar in a manner which causes operations to have to retreat past a pre-existing lift;
- maximise rate of retreat by minimising dead time associated with flitting (relocating) machines; and
- minimise intersections and the time spent working in the vicinity of intersections, both when retreating away from an intersection at the start of extracting a pillar and when retreating through an intersection at the completion of a pillar.

Modern pillar extraction methods have a focus on avoiding having to split pillars during secondary extraction and on minimising the number of intersections. Figure 8.26 shows an example of how the manner and sequence of extracting pillars can impact on the convergence distribution in and about the workplace. The convergence contours are specific to the particular stage in the extraction of each pillar. An important point to note is the extent to which convergence at and immediately outbye of the

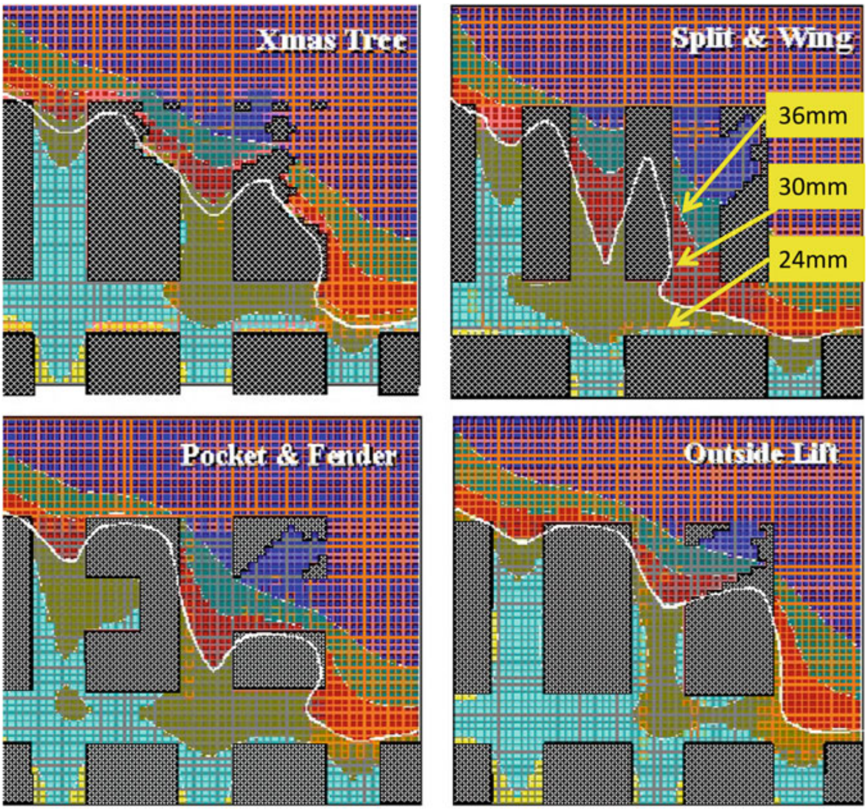


**Fig. 8.26** An example of the use of numerical modelling to evaluate how the manner and sequence of extracting pillars can impact on roof convergence in the vicinity of a workplace, with the white line corresponding to 30 mm predicted roof convergence (Adapted from Mark et al. 2003)

face can vary with the manner and sequence of extracting pillars.

The design of the manner and sequence of extracting pillars needs to have particular regard to fender stability and to intersection and stook behaviour, discussed in following subsections. This process requires knowledge of ground engineering principles and operational experience. For these reason, it is very important that variations to any approved plan showing the manner and sequence of pillar extraction are not made without referral to mine management and an onsite inspection by appropriately qualified management personnel. The history of pillar extraction contains numerous examples of serious accidents associated with deviations from the planned manner and sequence of pillar extraction.

### 8.4.3.4 Fenders

With the introduction of MRS units, there has been a trend towards the use of Wongawilli style extraction methods in massive and strong roof environments in some mining districts in order to restrict damage to the MRS units. These methods also continue to find application at depth since they avoid many of the stability problems associated with highly loaded standing pillars. Remote controlled continuous miners now provide the opportunity to use fenders up to 15 m wide in these layouts.

Historically, little consideration has been given to mining height, depth, nature of the superincumbent strata, stiffness of the loading system, and fender post-failure stiffness when designing fender width. Rather, width has been determined primarily by the need to keep operators under supported roof.

Up until the early 1990s, fenders were typically 6–7 m wide and it was generally considered that they were in a yielded or destressed state at the time of extraction (Hams 1974; Wardle and McNabb 1985). However, Shepherd et al. (1990) and Shepherd and Lewandowski (1994) concluded from stress monitoring conducted in the immediate roof above fenders and from convergence monitoring that fenders were not always in a yielded state and that they could experience a considerable increase in stress during lifting off. The researchers recommended a fender width of 9 m, which gave rise to considerable debate. Sleeman (1993) stated that to those who have had extensive operational experience in the highly stressed Illawarra coal measures in Australia, the results did not appear to readily fit field experience. This experience was mining conditions were better during pillar extraction than during panel development.

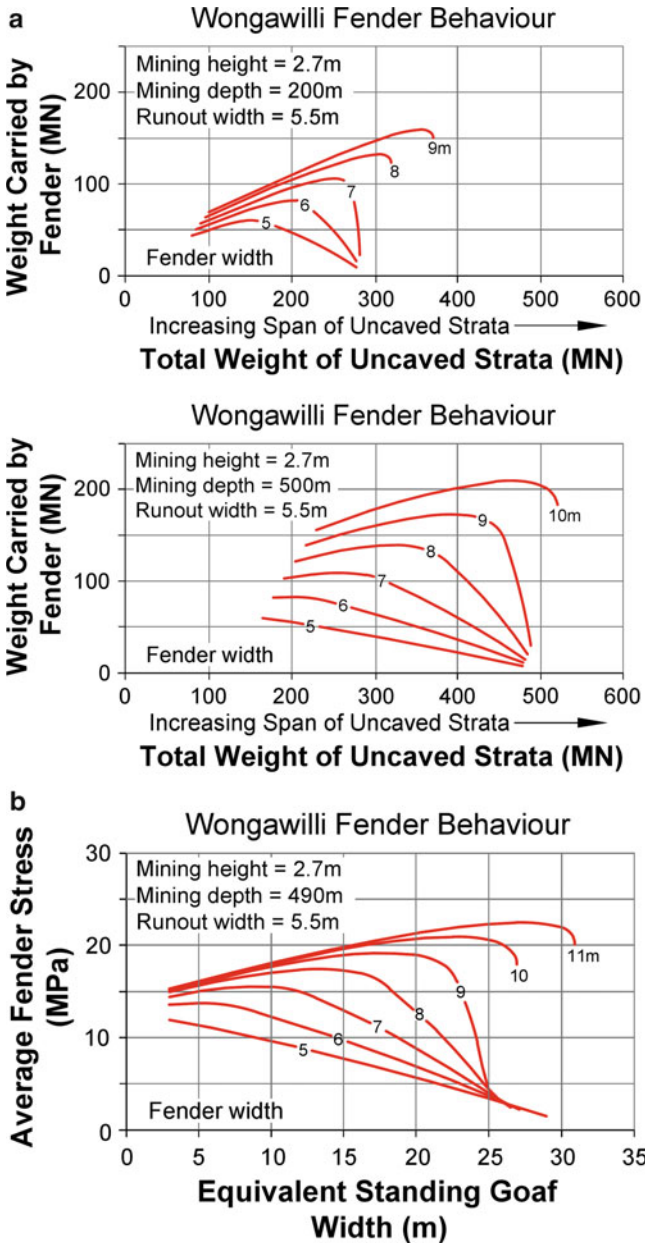
There is a range of reasons which could account for the different outcomes, notable amongst these being the unreliability of stress measurements and misconceptions as to what was actually being measured. Sleeman’s observation of better conditions during pillar extraction may have had more to do with the relief from horizontal stress provided by the formation of a goaf.

Advances in numerical modelling provide clarity. Figure 8.27 shows the significant influence of the stiffness of the mining system on fender behaviour in the case of Wongawilli pillar extraction at depths of 200 m and 490–500 m (Quinteiro and Galvin 1993, 1994). The mining layout and geological properties are identical in both cases. All overburden load has been assumed to be carried by the fender and the abutments; that is, there is no load transfer to the goaf. Fenders fail at the same load but the manner in which they subsequently shed load, or yield, is significantly different. As the depth of mining decreases, the stiffness of the roof strata reduces and so it is less capable of transferring load from the fender onto the panel abutments. In this softer system, the roof ‘chases’ the fender, causing it to yield more rapidly. The model shows that at a depth of around 200 m, fenders of less than 7 m width yield almost immediately upon drivage.

### 8.4.3.5 Intersections and Stooks

Experience confirms there is a substantially increased likelihood of a fall of ground as pillar extraction operations retreat through an intersection. A review of 44 incidents where continuous miners were immobilised by falls of ground during pillar extraction in NSW (Australia) identified that 64 % of the incidents occurred in

**Fig. 8.27** The influence of surrounding strata stiffness on the behaviour of goaf edge fenders. (**a**) Fender behaviour expressed in terms of load. (**b**) Fender behaviour expressed in terms of extraction span (After Quinteiro and Galvin 1993, 1994)



the vicinity of intersections during extraction of the first or last lift off a fender and that 50 % of the associated fatalities were persons simply waiting in intersections (Galvin et al. 1994). NIOSH (2010) reported that 13 of the 25 fatal pillar recovery incidents in the USA since 1992 involved falls of ground in active intersections, with three more occurring in intersections further away from the mining activity.

Intersections are prone to falls of ground because:

- the protection previously provided to the face operation by solid coal on one or both sides of the retreat roadway no longer exists (Fig. 8.28);
- there is a significant increase in the effective span of the exposed roof through which

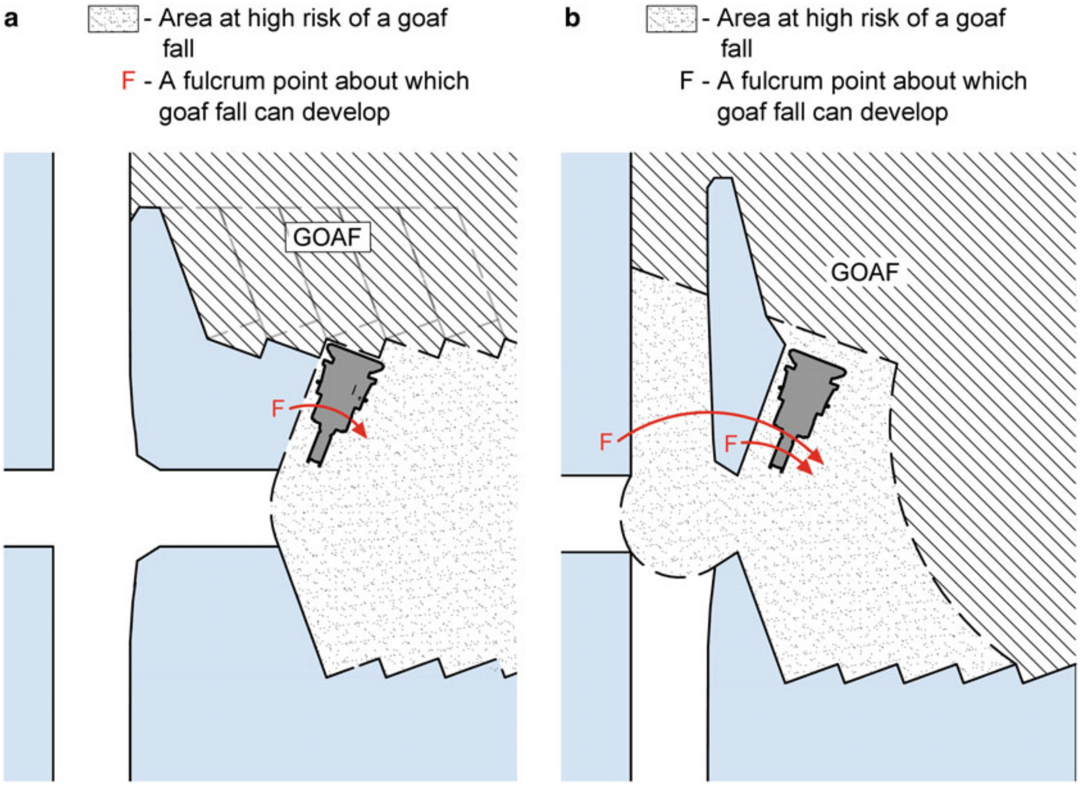

a
▨ - Area at high risk of a goaf fall
F - A fulcrum point about which goaf fall can develop
GOAF
F
b
▨ - Area at high risk of a goaf fall
F - A fulcrum point about which goaf fall can develop
GOAF
F
F

**Fig. 8.28** Conceptualisation of working face located beneath cantilevered roof at a goaf edge

operations have to retreat, resulting in the roof having a reduced load carrying capacity and increasing the potential for fracture development within the immediate roof horizon;

- the roof, ribs and floor of intersections may have been subjected to load in an unconfined state for an extended period, providing the opportunity for time dependent deformation leading to further reductions in load carrying capacity, increased fracture development and increased effective roof spans; and
- in the final stages of extracting a fender, the immediate roof of an intersection progressively reverts from being a quasi plate supported at four corners, to a long cantilever with its fulcrum located outbye of the intersection (Fig. 8.28).

As pillar extraction operations approach an intersection, each successive lift has an accelerated effect on the redistribution of load. This can be appreciated by applying Eq. 2.3 to a 33 m long fender that is extracted using 3.5 m wide lifts in the manner shown in Fig. 8.29. Extraction of the first lift reduces the stiffness of the remaining fender by 11 %, while taking the last lift to leave a 4 m wide stook results in a 47 % step reduction in fender stiffness. Hence, the same increase in fender load could be expected to produce more than a four-fold increase in fender compression when the last lift is taken. The situation is aggravated further by a significant reduction in the stiffness of the roof strata as the intersection is approached.

Thus, in the vicinity of an intersection:

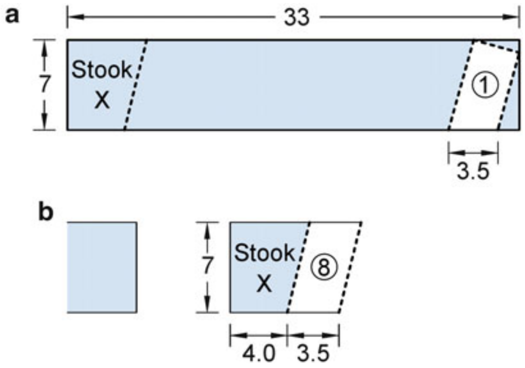


**Fig. 8.29** Illustration of accelerated change in fender stiffness with progressive extraction of the fender. (**a**) Fender stiffness changes by 11 % due to extraction of Lift #1. (**b**) Fender stiffness changes by 47 % due to extraction of Lift #8

- strata can be expected to behave in a significantly different manner to that in earlier stages of lifting off a pillar or fender; and
- small changes in extracted area can trigger large changes in load distribution and therefore, strata displacement.

The preceding factors contribute to a higher rate and severity of floor heave, rib spall, 'drummy' roof and roof falls at intersections, and to these instabilities developing quickly as extraction operations approach an intersection. It is important to implement prevention and mitigation measures ahead of pillar extraction to avoid these instabilities costing time during the final stages of extracting a pillar, when rapid access and egress and uninterrupted production are most critical to safe and efficient extraction. Measures, many of which are equally relevant to commencing to lift off a pillar at an intersection, include:

- designing panel dimensions to restrict abutment loadings;
- designing the panel layout to minimise intersections;
- providing for increased support density, which may include longer support tendons in intersections;
- designing the panel layout and extraction sequences to minimise breakaways (turnouts) and pillar corners trimmed to provide vehicle (haulage) access;
- restricting roadway width;
- driving roadways to design width;
- driving roadways on centre;
- installing additional support in a timely manner at known problem areas, preferably during development and certainly before the areas are subjected to abutment stress;
- maintaining roof support in good condition;
- utilising remote control continuous miners and MRS units;
- monitoring the face area and adjacent intersections frequently throughout the shift;
- installing standing support in an intersection as early as practical to reduce effective roof span and to 'tighten up' the intersection;
- restricting the first and last lifts off a pillar to single pass; and
- stopping extraction operations short of the intersection in order to leave a substantial stook as a temporary support.

It should be noted that in the case of pillar extraction in existing bord and pillar workings, the available options are restricted to secondary support and observational measures. This reinforces the importance of considering pillar extraction at the design stage of first workings.

Stooks offer one of the most effective forms of intersection control. This is because stooks:

- are formed in situ and, therefore, they provide continuous resistance to roof and floor displacement;
- have a considerably higher stiffness than all artificial forms of support, such as tendons, timber props and MRS;
- have a considerably higher load carrying capacity than all artificial forms of support (Table 6.1); and
- are positioned towards the centre of the intersection span, where maximum benefit is to be had from passive support.

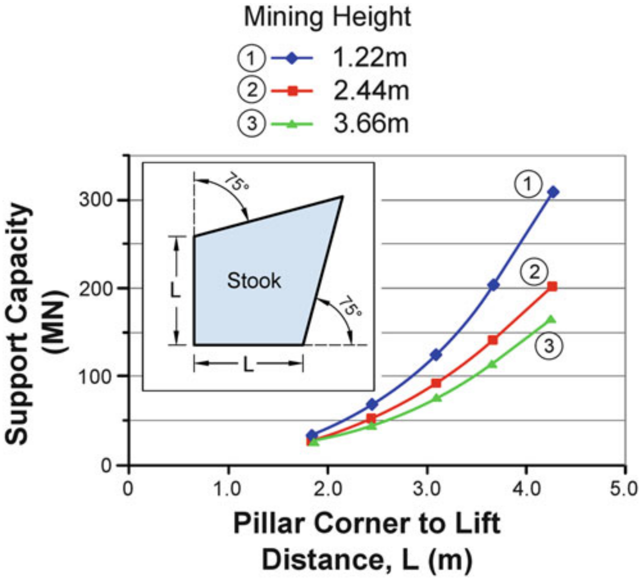


**Fig. 8.30** Load bearing capacity of a final stook of equal side lengths, L, from pillar corner and driven at 75° to pillar sides, based on the design procedure of Mark and Zelanko (2001)

Because stooks tend to be irregular in shape and of small width-to-height ratio, the calculation of their load bearing capacity falls outside the range of pillar strength formulae. Nevertheless, the Salamon and Munro formula (Eq. 4.23) and the Bieniawski-PSU formula (Eq. 4.29), for example, provide insight into stook strength. In the case of a 2.7 m high, 6 m long, 3 m wide stook unaffected by geological structure, the formulations predict a strength of the order of 110 MN (11,000 t) (Galvin et al. 1994). An alternative approach proposed by Mark and Zelanko (2001) and based on dividing a stook into a number of yield zones, produces the strength predictions shown in Fig. 8.30. Once again, stook strengths are in the thousands to tens of thousands of tonnes range. Irrespective of the accuracy of the analysis, the results demonstrate the superior load carrying capacity of a stook.

The substantial strength of a stook has long been appreciated by operators and has led to a range of views as to the impact stooks have on the development of caving and abutment stress. In South Africa, for example, leaving stooks or fenders, other than those of inconsequential size, has been discouraged. It has been common for pillar remnants to be blasted if they could not be recovered and, in some cases, wire rope slings have even been used to withdraw timber finger lines and breaker props as soon as they have performed their primary support function (Galvin 1993a). Australian practice has evolved to the other extreme, where leaving a stook has been standard practice and effectively mandatory, other than in a few partial extraction operations. This has contributed to incidents in strong roof conditions.

USA operators have had more discretion in deciding if and to what extent they extract the final stook in a lift. Mark and Chase (1997) reported that 49 % of the 67 roof fall fatalities in pillar extraction in the period 1978–1986 occurred during mining of stook X (or the final stump). Between 1982 and 2001, 21 persons were killed in 17 incidents associated with mining the final stook or lift (Mark and Zelanko 2001). NIOSH (2010) reported that traditionally, miners tried to extract all the coal during pillar recovery because they were concerned that stumps would inhibit caving and cause the outbye pillars to squeeze. However, recent experience indicated that fears about leaving stumps were exaggerated.

In the unlikely event that, in practice, a 2.7 m high stook with dimensions of 6 m × 3 m in plan did have an extreme strength close to 110 MN (11,000 t), the deadweight load of an area of only

8 m × 5.5 m is likely to be sufficient at a depth of 80 m to fail the stook. Hence, there is ample driving force available to fail large stooks. However, careful consideration has to be given to how far into the goaf the stooks need to be before this deadweight load is generated; what are the implications for abutment stress at the working face in the interim; what is the likely mode of failure of the stooks; and what are the implications of this failure.

The trend toward longer lifts utilising remote controlled continuous miners has caused stook practices to be reviewed, since it results in longer stooks that are substantially wider at their inbye end. In extreme cases, stook sizes can range up to 75 $m^2$ and might be considered to constitute pillars. The larger stooks offer little additional benefit in supporting the intersection but represent a serious impediment to caving. It has become standard practice, therefore, to take one or more lifts off the back of these large stooks from the adjacent cut-through, or crosscut (Fig. 8.31).

Hence, a stook needs to be designed to be sufficiently strong to function as an effective roof support until operations have retreated from the immediate area but weak enough to fail soon after, so as not to hinder caving. Generally, large stooks are not viewed as an impediment to caving in weak roof environments. However, they can have adverse implications for face stability. For example, Shepherd et al. (1990) reported that under weaker, laminated roofs with lower shear strengths in deep Australian operations, larger stooks offered too much support to the roof cantilever, causing the beam to fail in shear just over the solid edge. This resulted in ‘chase outs’ and potentially dangerous situations over the continuous miner. If the immediate roof falls around a large stook or even if the stook yields, it still has a high residual stiffness, so impeding caving and subsidence of the upper strata and attracting stress. This can have serious implications for subsequent mining operations in adjacent seams.

Large stooks are often implicated in having to abandon pillar extraction because of excessively high abutment stress, with extraction often recommencing a number of rows further outbye. In some cases they are the root cause of the problem and in others they are a symptom of another problem, this often being inappropriate panel span. Large stooks and remnant pillars may have to be left in the vicinity of locally disturbed ground to provide additional support and/or because the risk associated with extracting the remaining coal is too high. Poor physical mining conditions, such as water ponding, broken or boggy floor, or a seam roll may also prevent a pillar from being extracted to the full extent of

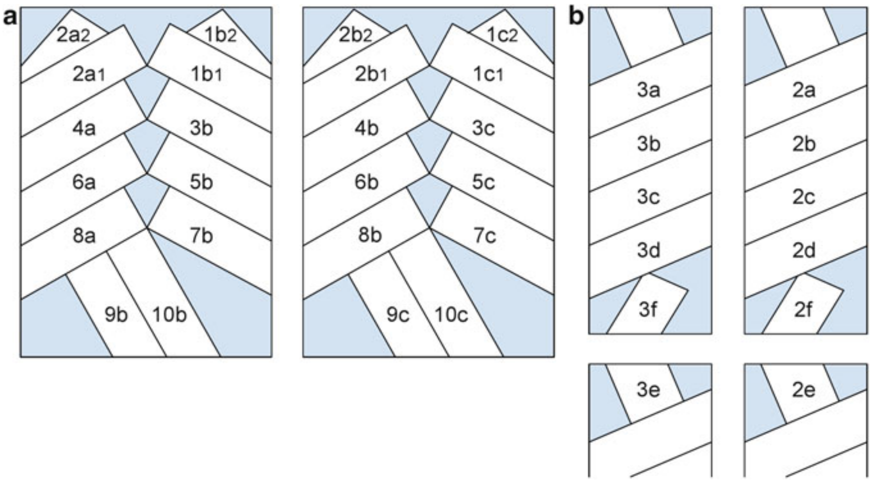


**Fig. 8.31** Examples of two lifting sequences for mining the outbye end, or back wing, of a pillar

the plan. Whatever the reason for leaving large stooks and remnant pillars, this is frequently the starting point for the development of hazardous ground conditions in total extraction panels. The situation is prone to become self-perpetuating and to increase in severity over time because the deterioration in ground conditions prevents a pillar from being fully extracted as planned and/or because a response to the poorer ground conditions is to leave larger stooks. Figures 8.23, 8.24 and A9.1 present examples of these circumstances.

The consequences of not fully extracting coal pillars tend to increase in severity with increase in spanning capacity, or competence, of the roof strata. In the case of more competent immediate roof strata, large stooks promote extensive strata overhang, which then causes a deterioration in strata conditions in and about the face line; promotes development of periodic weighting; and increases the risk of violent roof collapses and associated consequences. These consequences include the goaf over-running the breaker lines and entering the workplace, inrush of flammable or noxious gas and water, and windblast.

The risk presented by leaving large stooks or remnant pillars in total pillar extraction panels is increased significantly if they impede caving to the extent that the goaf remains open. These stooks effectively constitute pillars, typically having a safety factor of less than one and a width-to-height ratio of less than three. Hence, the conditions are set for an uncontrolled pillar collapse. As with localised delayed caving situations, windblast, gas inrush and goaf falls that override breaker lines and enter the workplace are all hazards associated with a sudden collapse of these pillars. However, the risk presented by these hazards is much greater in the case of a sudden pillar collapse.

As with any coal pillar, the behaviour of a stook is a function of the stiffness of the surrounding strata as well as that of the stook. The theoretical capacity to quantify the stiffness of the loading system in a total pillar extraction environment is limited. Analysis is made more complex if local roof falls result in an increase in stook height. Hence, the design of stooks is still highly reliant on local experience. This is an added reason for the careful and ongoing monitoring of the effectiveness of pillar extraction designs and the timely implementation of any remedial actions.

A mandated requirement in some jurisdictions to leave stooks has been a source of angst amongst some experienced operators when conducting pillar extraction under strong roof strata. It is a legacy of a past era when rigid controls were mandated in an attempt to prevent a reoccurrence of an incident. Given advances in technology, particularly the development of MRS and remote controlled continuous miners, complemented with a risk management approach to developing safe systems of work, this situation may be resolved through formal risk assessment on a site-specific basis. Guidance on undertaking risk assessment is provided in Sects. 12.3, 12.4 and 12.5. The need to conduct the risk assessment objectively with an appropriate skill set is essential. The risk assessment must lead to an informed objective outcome, rather than just going through the motions in order to justify a pre-determined desired outcome.

In addition to stooks, consideration needs to be given to the support installed in the intersection itself. Very often, the degree of intersection support is well below that associated with three way intersections in longwall mining, even though mining is taking place in, arguably, a more hostile environment.

### 8.4.4 Workplace Stability

Workplace stability is primarily concerned with preventing falls of ground outbye of the face and encroachment of the goaf into the workplace. Factors to take into consideration are:

- monitoring of roadway stability, especially intersections;
- installation of secondary support;
- control of goaf encroachment; and
- operating practices.

#### 8.4.4.1 Monitoring of Stability

All persons in a pillar extraction panel need to be particularly alert to changes in ground conditions. Coal haulage operators are in a good position to monitor for deterioration in stability and to trigger a response plan because they pass regularly and frequently through the abutment stress zone.

In some mining jurisdictions, it is standard practice to monitor intersection stability using inspection holes drilled some 10 or more metres into the roof. This type of monitoring is not continuous and movements can go undetected. It can also put operators at risk of being run over by haulage vehicles or being caught in intersection falls. It is preferable that extensometers are installed in these holes to enable roof displacement to be monitored more quickly and continuously from a safe position. Extensometers are a valuable additional monitoring aid for all members of the workforce but especially for haulage operators and supervisors.

#### 8.4.4.2 Installation of Secondary Support

Before commencing to extract pillars, the support installed in existing bord and intersections should be examined and the need for secondary support determined and responded to well in advance of pillar extraction operations. Situations that require secondary support to be installed close to the working face require careful consideration. Ongoing deterioration in conditions while extraction is stopped to install the support, the practical difficulties associated with installing support in a dynamic mining environment, and the exposure of operators to the risk of a fall of ground during the installation process all need to be considered. The risk might be too high to justify installing the support in lieu of pulling out of the area. Alternatively, the risk might be better managed by approaching the area of concern from a different direction or avoiding it altogether.

#### 8.4.4.3 Control of Goaf Encroachment

In most situations, the working face in a pillar extraction panel can be visualised as being located beneath a cantilevered beam of roof strata overhanging into the goaf, as depicted in Fig. 8.28. In an attempt to prevent the cantilevered strata snapping off on the outbye (solid) side of the workplace, it is an established mining practice to construct a fulcrum, or breaker line, comprised of timber props, rock bolts or MRS units immediately on the inbye side of the roadway leading to the active face. Each time a lift is completed and whenever operations retreat through an intersection, a new breaker line is established (Fig. 8.3).

Historically, breaker lines have comprised two or three rows of 4–6 substantial diameter (150–200 mm) timber props (Fig. 8.3). The number of rows of props is related not only to the support capacity required to break off the roof strata but also to spare capacity to compensate for props being dislodged when caving occurs. The overall support capacity of most timber prop breaker lines is substantially less than the combined support capacity of the individual props. This is because the props can be set to different standards and preloads and there can be considerable differences between their individual stiffnesses and also between the stiffnesses of their end foundations (solid rock, timber slab, loose coal, etc.).

Timber breaker lines offer the advantage of providing early warning of convergence and impending falls. However, they are heavy and require operators to work close to the goaf edge. They can take time to measure, cut and set, although an experienced crew will often predetermine their location and have them cut to length before the continuous miner withdraws from a lift. Effectiveness, manual handling, cost and speed considerations are resulting in timber breaker lines being complemented or replaced in many operations with tendon breaker lines and mobile hydraulic supports.

The concepts of rock bolt breaker lines and mobile roof supports (MRS) for goaf edge control in pillar extraction were proposed by Wagner and Galvin (1978) following the death of six mine workers when the goaf overrun timber breaker props at Vierfontein Colliery in South Africa in 1978. The first rock bolt breaker lines consisted of 1.2 m resin anchored bolts and were not totally effective, causing bolt length to be increased to 1.8 m (Flint et al. 1984). McCosh et al. (1989) trialled two rows of 1.8 m long, mechanically end anchored roof bolts with headboards as breaker lines, 5 bolts per row in a 6.0–6.5 m wide bord, to successfully extract 500 pillars from under sandstone roof in South Africa. It was found that the first row of bolts needed to be 0.5–2.0 m back from the goaf edge abutment, increasing to 4.0 m if the goaf was still standing after three rows of pillars had been extracted. Beukes (1991) noted other successful applications in South Africa.

Shepherd and Singh (1998) trialled rock bolt breaker lines under sandstone roof in NSW as a means of also controlling feather-edging (Sect. 12.2). Based on extensive instrumentation, it was considered advisable for bolted breaker lines to comprise two rows of 1.8 m long, fully encapsulated bolts, with a 0.3–0.4 m row spacing and an implied density of at least 4 bolts per row in a 5.5 m wide bord. Subsequently, some operations have increased tendon length by using cable bolts, with Marshall Miller and Associates (2006) expressing the view that cable bolts of proper length and installation are equivalent to a double row of breaker props (although the number of cable bolts was not stated). Tendon type breaker line supports offer the advantage over other breaker line support systems in that they can be installed well in advance of mining operations.

MRS units are an alternative to timber breaker props and rock bolt breaker lines. While a MRS has a significantly higher stiffness than a timber prop, it is nowhere as stiff as a coal pillar of similar bearing area. The support capacity of a MRS is often directly equated to timber props by dividing its support capacity by that of a timber prop. For example, a 6 MN (600 t) capacity MRS might be equated to 12 timber props of 500 kN (50 t) capacity. However, this underrates the support benefits of a MRS because it assumes that the props share load equally and all props load at the same rate.

MRS units are deployed in a range of configurations. It is standard practice in Australia to use two at the face line in single sided lifting and three in double side lifting. One or two additional units may be used when retreating through an intersection. In the USA, it is common practice to only use two units at the face when double sided lifting and to employ an additional two units at the intersection. When three MRS are used in double sided lifting, a niche, or stable, has to be cut at the beginning of the lifting off operations to provide sufficient width to accommodate the third MRS. This operation and the flitting of the third MRS into position consume additional time, as does recovery of the third MRS through the 'neck' of the split and across the intersection at the completion of lifting off a fender or pillar. However, three MRS provide additional security against the goaf running into the working place because they extend the length of the breaker line and ensure that two MRS are set to the roof at all times.

The effectiveness of MRS units is highly dependent on their location relative to the workplace and on their mode of operation. They should be clustered about the centreline of the split to take advantage of the pre-supported roof and be set as close as possible to the continuous miner without impeding its passage into and out of the lift. A sufficient distance, typically >300 mm, needs to be maintained between each support to avoid physical interference between them. Figure 8.32 shows a spiling operation from an adjacent split to recover the three MRS units shown in Fig. 8.14. These were buried close to an intersection when two of the MRS units could not be advanced because their side plates had become interlocked. When lifting left and right at some Australian operations, the MRS units may be spread up to 4 m apart to cater for the increased span.

When advancing MRS units at the face, it is important to do so alternatively and to restrict maximum advance to half a canopy length. In friable conditions, it is good practice to lower the rear legs first to allow loose material to drop into the goaf rather than to fall in front of the machine and impede its advance out of the goaf. Appendix 10 provides a more comprehensive list of the advantages and disadvantages of MRS units and

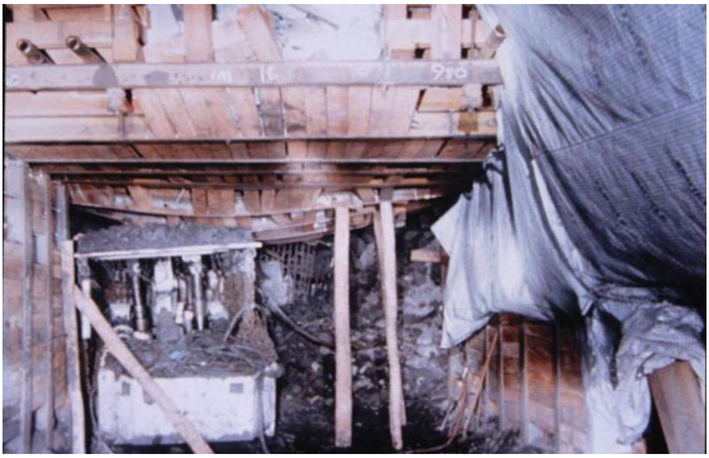


**Fig. 8.32** Spiling through goaf accessed by driving a split at right angles through a fender in order to recover three MRS units buried in a roof fall in the previous split

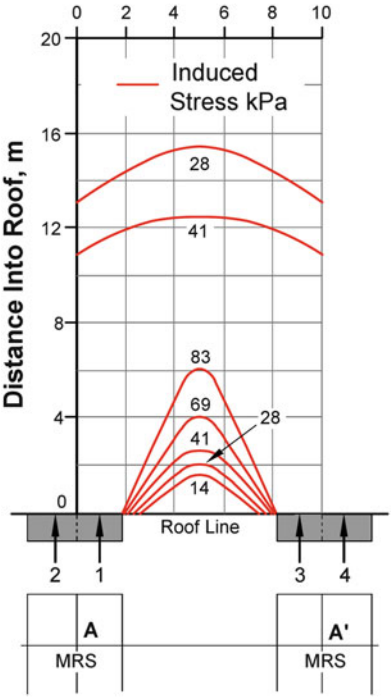


**Fig. 8.33** Isobars of stress induced in the superincumbent strata by two pairs of MRS set 6 m apart (Adapted from Maleki et al. 2001)

factors to consider when developing standard work procedures for their operation.

An issue arising from accidents involving persons working in and around MRS units concerns the area of influence of a MRS unit and its effect on the immediate roof. Maleki and Owens (1998) used analytical solutions to study the area of influence of 2 pairs of MRS set 6 m apart (Fig. 8.33). The analysis predicted that the units generated more than 40 kN (4 t) of upward force per square metre some 12 m up into roof between the midpoint of the pairs of MRS. However, the MRS units provided very little support to the first metre or so of immediate roof between the two sets of supports, acting instead as abutments to reduce the span of the bridging immediate roof strata. It should be assumed, therefore, that a MRS unit does not provide any support to the immediate roof around the MRS when this roof is fractured and when there is no abutment in close vicinity for this strata to bridge if it is not fractured.

#### 8.4.4.4 Operating Practices

As the strength of rock under load can reduce over time, the rate of extraction is a very important consideration for maintaining ground control in pillar extraction, with continuity of extraction particularly important during each cycle of extraction across a panel. Where practical, pillars should not be pre-split outbye of the extraction line and, preferably, only immediately prior to extraction. They should not be left in a pre-split state prior to an extended shutdown of the mine (exceeding say, a weekend). Partially extracted fenders should be completely extracted prior to a weekend shutdown or other extended non-production period. Operators need to be aware that the onset of caving can be less predictable at shallow depth and can also be associated with an increased risk of windblast (Sect. 3.5).

## 8.5 Operating Discipline

Pillar extraction is a mining method that still relies heavily on judgements based on experience and is relatively intolerant to errors, often with high potential consequences. A management plan is essential for the design, implementation and control of pillar extraction operations. The elements of such a plan are detailed in the *Manual on Pillar Extraction* (MDG-1005 1992). The relevance of a number of the principles of pillar extraction noted in the manual and in this chapter are illustrated by incident safety alerts such as those issued by regulators (for example, NSW Dept. Mineral Resources (1998a, b) and MHSA website).

One of the major risks associated with pillar extraction is that conformance to design is highly dependent on operating discipline. There are few inbuilt engineering controls to prevent deviation from plan and the effectiveness of most controls can be jeopardised by such deviations. Success is highly reliant on soft controls in an environment where a small change in physical conditions or practice can have a major impact on safety, stability and production. Hence, pillar extraction operations need to be premised on well thought out and risk assessed procedures underpinned by training, competency assessment, monitoring and regular review for effectiveness. Operations have to be supported with strong management and supervision.

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# Longwall Mining

# 9

**Abstract**

Since the early 1980s, the longwall method has developed into the safest, highest producing and most productive form of underground coal mining, rivalling the performance of many surface mining operations. This is due in large part to the rapid uptake of computer based technologies for automation and monitoring; improved reliability and performance of longwall mining equipment; and the adoption of plant management and loss control principles. This situation has many important implications for the geoscience and geotechnical engineering professions. For example, lost opportunity costs associated with loss of ground control are now so high that many of the simple observational and empirical approaches traditionally applied to geotechnical designs and operational aspects in longwall mining are no longer commensurate with the business risks that have to be managed. There is an increased need for geotechnical input to be based on sound engineering principles that encapsulate measured ground behaviour, applied mechanics, and numerical modelling. Ongoing research is important to support this need.

This chapter addresses geotechnical principles and practices relevant to satisfying these engineering requirements, making extensive use of figures and photographs to illustrate important concepts. It considers panel layout options and associated chain pillar design; traces the history of powered support design to draw learnings about their static and kinematic requirements; identifies and assesses operational variables, including cutting and support techniques, powered support maintenance, and face operational practices. It then reviews face behaviour and ground control requirements and practices; and evaluates the design and support of installation roadways and longwall recovery roadways, including pre-driven roadways.

**Keywords**

Abutment loading • Bidirectional cutting • Caliper shield • Canopy ratio • Chain pillar design • Chain pillar function • Chock shield • Conventional mode • Cutting technique • Cyclic loading • Double chock • Floor heave • Gateroad orientation • Ground control • Ground response curve • Half web

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cutting • Horizontal stress • Lemniscate linkage • Longwall • Longwall installation roadway • Longwall recovery • Longwall top coal caving • Miniwall • One web back • Periodic weighting • Powered support • Powered support hydraulics • Pre-driven roadway • Sacrificial roadway • Shield • Skew roof • Strata control • Stress notch • Stress relief • Support resistance • Tailgate abutment loading • Tip-to-face • Unidirectional cutting • Yield pillar

## 9.1 Introduction

Since the early 1980s, longwall mining has developed into the safest, highest producing and most productive form of underground coal mining, rivalling the performance of many surface mining operations. In Australia, for example, advances in technology, geotechnical engineering and work practices over that period have resulted in more than a three-fold increase in the average productivity of longwall mining, with some newer operations achieving up to an eight-fold increase. Subject to adequate coal reserves, environmental constraints, and access to capital, it is the method of choice for new underground operations.

The significant increases in longwall productivity are due in large part to rapid uptake of computer based technologies for automation and monitoring; improved reliability and performance of longwall mining equipment; and the adoption of plant management and loss control principles, leading to both increased rates of production and reduced labour requirements. In terms of production, average daily output of the top performers in Australia has increased from, typically, 5,000 t/day in 1985, to 20,000 t/day in 2013. Incremental cost savings associated with these rates of production, supported by increased coal selling prices, meant that delayed profit opportunity for each day of unplanned stoppage of a longwall face increased from as little as A$10,000/day in 1985, to in excess of A$500,000/day for some premium coking coal operations in 2013, with non-recoverable fixed costs sometimes being of the order of A$150,000/day.

This has important implications for the geoscience and geotechnical engineering professions. Firstly, operations are now in a better position to justify the engagement of geotechnical professionals, with the annual salary of one such person being recouped if their input avoids just one or two days of lost production per annum. Secondly, the lost opportunity costs are now so high that many of the simple observational and empirical approaches traditionally applied to geotechnical designs and operational aspects in longwall mining are no longer commensurate with the business risks that have to be managed. There is an increased need for geotechnical input to be based on sound engineering principles that encapsulate measured ground behaviour, applied mechanics, and numerical modelling. Ongoing research is required to support this need. This chapter addresses geotechnical principles and practices relevant to satisfying these engineering requirements.

## 9.2 Panel Layout

### 9.2.1 Basic Longwall Mining Methods

There are two basic types of longwall mining, namely, ‘longwall mining on the advance’ and ‘longwall mining on the retreat’. Longwall mining on the advance involves developing the maingate and tailgate entries just ahead of the longwall face as it is being advanced, with these gateroads being maintained in the goaf of the panel using various combinations of pack walls and arch support systems. The primary advantage of the method is that the single entry gateroads are located in stress relieved zones. Disadvantages include slow mining rates due to gateroad advance and longwall face advance being interdependent; restricted access for ventilation and supplies; ongoing roadway maintenance requirements; increased propensity for spontaneous combustion due to air ingress into the goaf; and no second independent means for egress in an emergency situation.

Longwall mining on the retreat is the most common type of longwall mining. It involves driving one, two or three gateroads down both flanks of a panel to its extremity, and then connecting these two sets of gateroads (Fig. 2.3). The longwall equipment is installed in this connecting roadway and the block is progressively extracted on the retreat. The method is not subject to many of the impediments associated with longwall mining on the advance and has a lower exposure to others. However, serious ground control difficulties can be associated with supporting and maintaining gateroads, cut-throughs and pillar ribs. Longwall mining on the retreat finds extensive application in Australia, South Africa and the USA at depths ranging from as low as 15 m, down to around 700 m. In these countries, it is premised on multiple entry longwall development that requires leaving one or two rows of interpanel pillars (chain pillars) between longwall panels. The method finds application using single entry gateroads at depths exceeding 1,200 m in Europe. It is also used extensively in China, including to recover coal from the goaf in thick seams (see Sect. 9.9.1).

The number of gateroads utilised in longwall mining is a function of many factors including egress requirements, ground conditions, gas and ventilation regimes, panel dimensions, production rate and propensity to spontaneous combustion. Three gateroads are always required when there is a requirement for two independent means of egress from the mining face. High gas regimes may also require three gateroads in order to provide a sufficient quantity of air to dilute the gas to safe and prescribed levels. Gas pre-drainage does not necessarily remove this requirement because rib emissions in the gateroads can still result in gas content in intake airways exceeding permissible levels (typically, no more than 0.25 % $CH_4$ equivalent in intake roadways) before the air reaches the mining face. This situation is aggravated in long panels, some of which can exceed three kilometres, due to the increased roadway surface area.

In any case, air quantity requirements at the working face may require three gateroads in very long panels in order to compensate for reduced air flow due to increased resistance associated with surface friction and shock losses. The relationship between ventilation pressure, air quantity and airway resistance is given by Eq. 9.1:

$$P = RQ^2 \tag{9.1}$$

where

$P$ = *fan pressure* (pa)
$Q$ = *air flow quantity* ($m^3/s$)
$R$ = *roadway resistance* ($Ns^2/m^8$)

The installation of standing support in roadways, especially tailgates, increases the resistance of the ventilation circuit considerably and can make the difference between requiring two or three gateroads in order to deliver the required quantity of air to the face. This is especially the case in coal seams prone to spontaneous combustion, as increasing fan pressure to compensate for increased airway resistance encourages air leakage between intake and return airways and across goaves, thereby promoting the development of spontaneous combustion. The basic principles for managing spontaneous combustion in these situations are presented in a range of literature, including Humphreys and Richmond (1986), Cliff et al. (1996) and MDG-1006 (2011).

Production rate also has a significant bearing on the number of gateroads required for longwall production. Gas emissions, dust make and heat generation increase with rate of retreat and, therefore, a higher quantity of air at an adequate velocity is required to safely manage these factors. The trend towards wider and higher longwall faces and higher capacity longwall production equipment also has a significant effect on heat production at the working face, with the power requirements of some installations exceeding 6 MW. Where conditions permit, consideration can be given to constructing a small diameter shaft at the inbye end of each panel as an alternative to driving extra gateroads for ventilation purposes.

Multiple gateroads aggravate ground control difficulties in longwall mining because the second and subsequent gateroads are exposed to longwall abutment stress. This necessitates that these roadways are either sufficiently remote from a

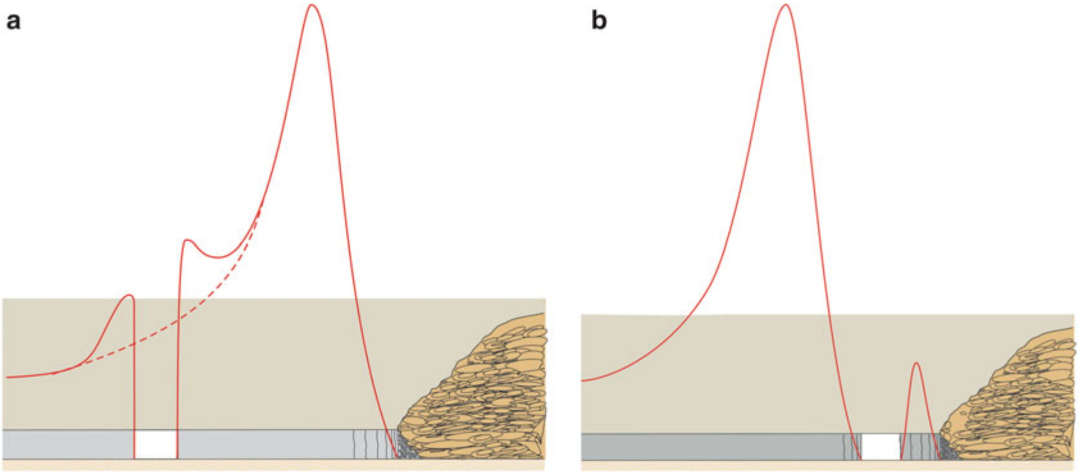


**Fig. 9.1** Schematic options for locating twin entry gateroads within an abutment stress zone. (**a**) Wide chain pillar in order to locate tailgate away from high abutment stress, (**b**) Very narrow chain pillar in order to induce controlled pillar yield so that gateroads are then located in a stress relieved zone

longwall panel that abutment stress impacts on them can be safely and productively managed or else, in the case of a twin gateroad situation, the second maingate roadway is located in the yield zone of the abutment stress profile. These options, illustrated in Fig. 9.1, determine the type and width of the chain pillars. Hence, chain pillars can range from squat pillars in the former case to yield pillars in the latter case.

A raft of additional ground control difficulties can be experienced in driving and supporting cut-throughs. Problems can arise on development when preference is given to orientating headings rather than cut-throughs in the more favourable direction for managing horizontal stress. They can arise on extraction because the cut-throughs are located within the abutment stress front. For these reasons and in order to minimise roadway drivage, the distance between cut-throughs is usually maximised, thus resulting in the length of chain pillars typically being two to five times their width.

Many of the basic ground engineering principles relevant to designing roadways, pillars and support systems in these circumstances are presented in Chaps. 2, 3, 4, 5, and 6. Aspects of these specific to longwall interpanel pillars (chain pillars) are developed in more detail in this chapter.

### 9.2.2 Gateroad Direction and Layout

Factors which need to be considered in optimising gateroad direction and layout include surface constraints; lease boundaries; coal quality consistency; coal thickness consistency, dip and dip direction; cleat intensity and direction; and horizontal stress magnitude and direction. Conflicts between these factors require design compromises, with high horizontal stress tending to be the most important and dominant factor controlling design. Figure 9.2 shows the three general design options for managing this factor.

For the purpose of this text, the layout shown in Fig. 9.2a is referred to as a 0/90/90 layout, indicating that the gateroads are orientated parallel to the major horizontal stress direction and the cut-throughs and longwall installation face are at right angles to this stress direction. This layout minimises the adverse impact of horizontal stress on the gateroads and maximises it on the cut-throughs and installation face. Therefore, it has the advantage of optimising conditions during longwall retreat at the expense of potentially difficult development conditions when driving cut-throughs and the longwall installation roadway. Conditions can be particularly adverse at the point of backholing a cut-through that has been developed from both directions.

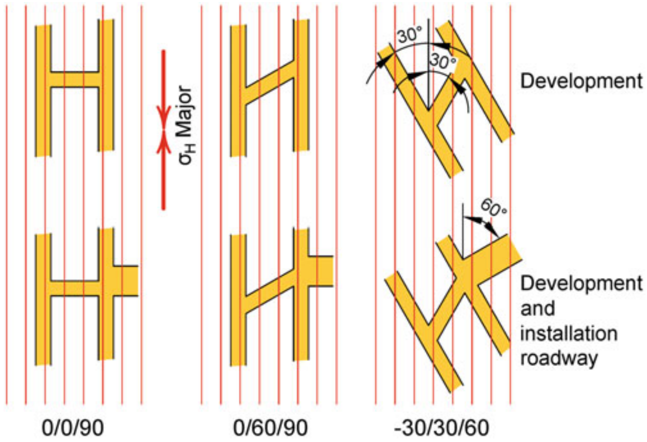


**Fig. 9.2** Orientation options for longwall gateroads, cut-throughs and installation roadway in a high horizontal stress field

Figure 9.2b depicts a 0/60/90 layout in which the gateroads continue to be orientated in the optimum direction and the installation face in the most adverse direction, but the cut-throughs are now orientated at 60° to the horizontal stress field to mitigate its impacts on them. The benefits of this option have to be weighed up against a range of operational impediments that can be associated with angling cut-throughs in this manner. These include:

- Cut-throughs can only be driven from one direction. This introduces scheduling restrictions and reduced operational flexibility, both of which may retard advance rates.
- Mobile plant cannot turn left and right into and out of cut-throughs, again reducing operational flexibility.
- There is a higher likelihood that one mining direction will not be optimum for controlling the impact of cleating and jointing, thereby elevating the risk of injury and causing an increase in mining spans due to rib spall.
- Two corners of the chain pillars are acute, resulting in them having a reduced strength, being susceptible to damage by equipment, and prone to fall along cleat and joint planes. In turn, these factors elevate the risk of injuries due to risk of rib fall and result in increased intersection spans and, hence, exposure to roof control problems.

The compromise situation of a −30/30/60 layout, in which horizontal stress impacts on headings, cut-throughs and the installation roadway are all reduced but not eliminated, is depicted in Fig. 9.2c. This layout can be difficult to implement due to other competing mining considerations and constraints, such as lease boundaries, seam dip, and gas management.

Other stand-alone or complementary options for managing horizontal stress in roadways during gateroad drivage and longwall extraction include primary and secondary support strategies and stress relief roadways. These aspects are discussed in more detail in Sects. 3.4 and 5.2 and Chaps. 6 and 7.

### 9.2.3 Chain Pillar Life Cycle

Longwall interpanel pillars, or chain pillars, can perform a variety of functions, some of which change over the life of the pillars. Essentially, the pillars are required to remain structurally stable and functional until at least the passage of the second longwall. In designing a chain pillar, consideration should be given to its life cycle, which can be broken down into five stages on the basis of pillar loading (Fig. 9.3). The stages are described in respect of a two heading development but the principles also hold true for a three heading development.

- Stage I. Two gateroads are developed into what, for practical purposes, could be considered virgin ground. Pillars are surrounded by solid in all four quadrants (that is, for 360°).

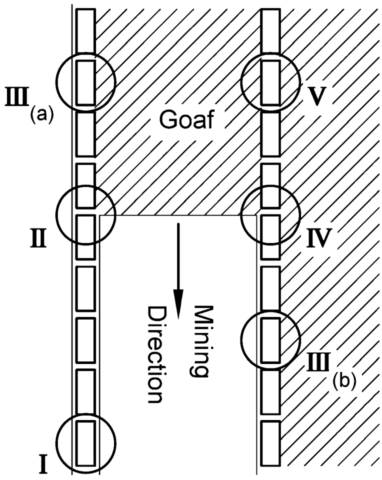


**Fig. 9.3** Loading stages in the life cycle of a chain pillar

Unless the depth of mining is very shallow, the overall width-to-depth ratio, W/H, of the gateroad panel is too small to result in full tributary load acting on the chain pillars. Depending on depth and horizontal stress magnitude and direction, the goaf of an adjacent longwall panel may cause the gateroad development panel to be shielded to some extent from horizontal stress.

- Stage II. The longwall face reaches the chain pillar and the pillar is now surrounded by solid in only three quadrants (270°). The abutment load of the goaf in the fourth quadrant is distributed between the longwall block and the side abutment, which includes the chain pillar, resulting in an increase in chain pillar load.
- Stage III(a). As the longwall retreats past a chain pillar, the abutment load carried by the longwall face is progressively transferred onto the chain pillar, or side abutment. Ultimately, the side abutment falls beyond the influence of the longwall face and the chain pillar is subjected to full **maingate abutment loading** arising from the goaf, which now occupies two adjacent quadrants of the pillar (180°). This situation is referred to as **single sided abutment loading**. The magnitude and distribution of the abutment stress profile and the width of the chain pillar determine the proportion of abutment load carried by the chain pillar.
- Stage III(b). The chain pillar occupies the same relative position as in stage III(a) and so, in theory, the abutment loading acting on it should be unchanged. However, because the strength of rock can decrease over time, especially under high load, the stability of both the chain pillar and the adjacent gateroad (tailgate) may deteriorate in the time interval between the passage of the two longwalls. Ongoing mining-induced changes in the stress field can also contribute to this deterioration.
- Stage IV. The chain pillar is now surrounded by solid in only one quadrant (90°). The abutment load of the second goaf is distributed between the second longwall block and the chain pillar, resulting in a further increase in chain pillar load. This situation is sometimes referred to as **tailgate abutment loading**.
- Stage V. As the second longwall retreats beyond the chain pillar, the abutment load carried by the longwall face is again progressively transferred onto the tailgate side abutment, with the chain pillar now surrounded by goaf in all four quadrants. This situation is referred to as **double sided abutment loading**. Due to the stiffness of the superincumbent strata, the load acting on a double abutment loaded chain pillar may not initially be double that for a single sided abutment loading situation, especially at depth. A number of additional panels may have to be extracted before this state is reached, as evident from the profiles of vertical surface displacement depicted in Figs. 3.16 and 3.19.

### 9.2.4 Chain Pillar Design

Chain pillars constitute interpanel pillars, with the basic principles pertaining to their function and design introduced in Sect. 5.2. There is no single correct design method for longwall chain pillars, particularly since the roles of chain pillars in a mine layout may be quite diverse (Hebblewhite and Galvin 1996). Nevertheless, in nearly all cases, a primary function of chain pillars is to provide a buffer of sufficient width between the goaf of the previous longwall panel and the gateroads of the current longwall panel in order to shield the gateroads from high abutment stress. Therefore, chain pillar design should include consideration of the abutment stress that gateroads can tolerate, having regard to the local geology, the type and density of support to be installed in the gateroads, and the level of serviceability required of them (Galvin et al. 1982).

A variety of empirical and numerical approaches are currently utilised to design chain pillars. Many of the empirical approaches rely on the concepts of angle of break and abutment angle and a single sided abutment load multiplying factor to estimate pillar load at the tailgate/face corner. This load is then compared to pillar strength calculated using an empirical equation derived for bord and pillar mining situations. The two most common means of calculating chain pillar width utilising these empirical approaches are:

- To work backwards from a design pillar safety factor that is judged to produce a safe working environment, acceptable tailgate conditions and, where required, adequate surface subsidence control. In some circumstances where only minimal surface subsidence is tolerable, UNSW power safety factors of 2.2 or more based on double sided abutment loading have been used with the intent of preventing pillar failure in the long term. Otherwise, the design safety factor is usually based on the notion of preventing pillar failure until after the second longwall face has passed by the pillar. in which case the maximum pillar load is taken to be that acting on the chain pillar at the tailgate end.
- To work backwards from a design stability factor selected on the basis of its empirical relationship to some measure of tailgate serviceability. The stability factor is equated to the ratio of chain pillar strength to chain pillar load, with the latter usually estimated at the tailgate corner. A number of permutations of abutment angle and single sided abutment loading multiplication factors can be associated with the calculation of pillar load in some approaches, for example, ALTS (M. G. Colwell et al. 1999). The philosophy of NIOSH (2008) in respect of ALPS and ARMPS needs to be borne in mind, this being that since these are empirical models derived from real-world data, they do not require a full understanding of the mechanics of pillar behaviour. This is an important consideration when applying the formulations at sites other than from where the data was sourced. Risk is always associated with situations where there may be a lack of understanding of the mechanics underpinning behaviour or where loading conditions are significantly different to the cases in the underpinning databases (for example, in multiseam mining situations).

Limitations are associated with both empirical approaches. For example, the concept of an abutment angle does not reflect the mechanics of overburden behaviour as depth increases (Sect. 3.2), while none of the empirical pillar strength formulae applied in the various design procedures were derived on the basis of the behaviour of pillars that abutted caved ground or for pillars in the high width-to-height ratio range of many chain pillars.

In Sect. 5.2.2 it was noted that numerical modelling has been promoted for designing chain pillars since at least the early 1980s. Nevertheless, limitations can still be associated with these approaches, especially in regard to quantifying pillar load, the effect of caving on pillar strength, and goaf reconsolidation characteristics. Notwithstanding this, the cost of undertaking parametric and sensitivity analysis utilising sensible numerical models in order to give confidence to chain pillar design is minor to trivial in comparison to the adverse safety, productivity and financial risks associated with a poor chain pillar design in longwall mining. The various ways in which numerical modelling finds application to chain pillar and gateroad design and support are reflected, for example, in the approaches of Salamon (1991), Gale (2004), Peng (2008), and Esterhuizen et al. (2010b).

As the depth of mining increases, strength considerations result in an increase in chain pillar width-to-height ratio. This has implications for both the pillar width required to provide an adequate buffer from abutment stress and for the propensity for pressure bursts within the chain pillars. A situation is also reached where, irrespective of the width of the pillar, induced stress levels at the pillar ribsides result in deformations sufficient to threaten safety and the serviceability of the gateroad. Longwall mining on the advance is uneconomic for mitigating these impacts. Hence, the concept of **yield pillars** has found application in designing chain pillars in deep longwall retreat operations in attempts to ameliorate pressure bursts, severe rib spall, and pillar punching of the roof and floor strata. The concept is also used in the USA to minimise coal sterilisation and to provide optimum geometries for place changing in three heading developments, and in South Africa to provide more uniform surface subsidence profiles.

The concept of a yielding coal pillar is based on the controlled unloading of a coal pillar once its peak load carrying capacity has been exceeded. It has been applied in the USA in two, three and four heading gateroad layouts. It relies on utilising the post-failure strength of a yielded pillar to provide local ground support, while transferring (shedding) the majority of the overburden and abutment load to adjacent, stiffer, non–yielding pillars. The terminology is sometimes confused, with a yield pillar also referred to as a **crush pillar**. Hebblewhite and Galvin (1996) report that many so-called yield pillars are, in fact, stable load-bearing pillars of very low height in benign roof strata conditions. It is important to appreciate the distinction since the penalty for poor design is severe in the form of sudden and unpredictable pillar collapse.

A review of gateroad yield pillar design approaches and applications in USA longwall operations by NSA Engineering (2000) found that yield pillars at that time were generally 6–9 m wide and ranged in width-to-height ratio from 3 to 5. No 'entirely successful' yield pillar designs were achieved when width-to-height ratio exceeded 5, nor were any 'operationally successful' full-yielding gateroad systems achieved in ground where the CMRR was less than 50. Measurements suggested that full yielding of a pillar seldom occurred until after the first adjacent panel has been extracted well outbye of the pillar and the majority of peak side abutment stress has been attained. Badr et al. (2002) reported similar findings, noting that previous designs have enjoyed mixed success and that load shedding requires three criteria to be satisfied, namely:

- there are load bearing areas (unmined seam or compacting goaf) nearby which can sustain the transferred load;
- the roof and floor are sufficiently competent to facilitate the load transfer without debilitating roof falls or floor heave; and
- the stiffness of the surrounding rock mass is sufficiently high to ensure that the equilibrium of the rocks remains stable.

Salamon et al. (2003) undertook numerical simulation of longwall chain pillars of width-to-height ratios 3, 5 and 10 at a depth of 700 m. The authors noted that their discussion of results deliberately avoided the quantification of the terms 'narrow', 'intermediate' and 'squat'. The research indicated that for narrow pillars, pillar deformation is controlled; the yielding zones progress towards the centre of the pillar smoothly; and a pillar that is yielding throughout its width can readily be created. If the depth of mining is great, this full yielding state can be reached during the development of the gateroad entries. The desktop analysis concluded that such narrow pillars make ideal yield pillars, their only shortcoming being that their load bearing capacity is low. This limits the spans over which they should be applied.

The study concluded that it appears pillars with an intermediate width-to-height ratio cannot be brought into a fully yielding state because their failure process becomes unstable when yielding penetrates to a certain depth. This critical depth could be reached either during primary development or during secondary longwall extraction. The instability may induce a pressure burst-like event and even a total collapse of the pillar. Therefore, the utilisation of yield pillars of this size should be restricted to relatively shallow cover.

Salamon et al. (2003) also concluded that squat pillars have the potential for sudden seismic events in their outer zone but, because of the large width of the remaining inner core, the rubble around the pillar sides provides sufficient confinement to enable stability to be re-established. These pillars were not considered ideal chain pillars in deep longwall mining situations.

A feature of most successful yield pillar and crush pillar outcomes to date has been the presence of very stiff immediate roof strata. This is not surprising, as the high stiffness of this strata regulates both the magnitude and rate of load transfer to pillars adjacent to the longwall block, thus controlling the rate of yield and failure mode of these pillars.

## 9.2.5 Chain Pillar/Gateroad Behaviour

### 9.2.5.1 Stage I – Development

During gateroad development, roadway and pillar behaviour are governed by the same principles that apply to bord and pillar mining. The main difference between bord and pillar main development and gateroad development, which is unlikely to be detectable in practice, is that the load acting on the gateroad pillars may be lower because the narrower gateroad panel width results in a smaller reduction in the stiffness of the overburden. Nevertheless, this load can still be expected to result in extensive fracturing of gateroad sidewalls as depth of mining increases.

In the case of intermediate depth longwall panels, actual pillar stresses may be comparable to those encountered at the shallower depths of typical bord and pillar mining. In deeper longwall situations, pre-mining rock stress will already be higher than pillar stresses normally encountered in bord and pillar mining and, therefore, extensive fracturing of gateroad sidewalls can be expected even at narrow panel widths.

### 9.2.5.2 Stage II – Maingate/Face Corner

As the goaf approaches a chain pillar at the maingate face corner, the pillar is subjected to increased abutment load. The presence of the goaf also causes a change in the state of horizontal stress in the immediate roof and floor of the gateroad. Primary factors that determine the magnitude of this change include the direction of the major horizontal stress relative to the gateroad direction; the composition of the immediate roof strata; the elastic modulus and Poisson's ratio of the immediate roof strata; and whether caving develops at the face or is delayed. Figure 9.4 shows an example of the manner in which a horizontal stress notch developed around the maingate end of a longwall face when the in situ major horizontal stress was approximately twice the primitive vertical stress high and orientated at around 30° to the maingate.

A widely-employed relationship between the horizontal stress concentration factor and the angle of the maingate to the major horizontal stress direction is plotted in Fig. 9.5a. It is based on stress measurements in stone strata some 5 m over the rib of chain pillars and 2.5 m above the mining horizon. Horizontal stress is shown to peak at almost 2.2 times primitive (virgin) stress at this horizon when the gateroad is orientated at approximately 65° to the major horizontal stress direction. The end user of the relationship shown in Fig. 9.5a needs to be cognisant that it is based on limited data, with the shape at higher stress angles being determined by just one data point (Cook Colliery). Furthermore, it has no regard to the impact on ground behaviour of the ratio between the two principal lateral stress directions.

An update of this relationship developed by Gale (2014) is shown in Fig. 9.5b. This revision has regard to some new data, face position and numerical modelling. Both relationships shown in Fig. 9.5 indicate that maingate stress conditions will be optimised when the gateroads

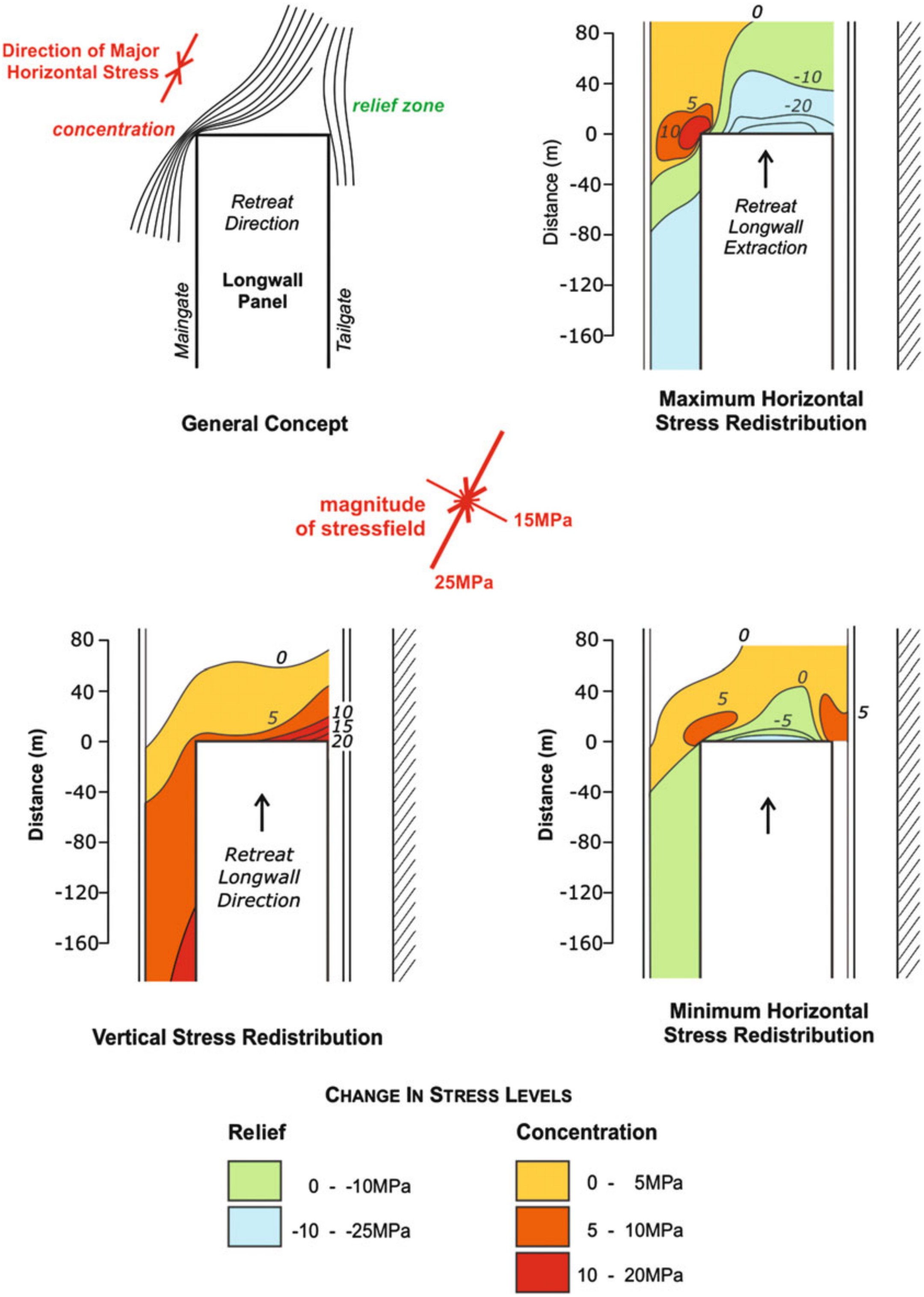

Direction of Major Horizontal Stress
concentration
relief zone
Retreat Direction
Longwall Panel
Maingate
Tailgate
General Concept
0
80
40
0
-40
-80
-120
-160
Distance (m)
-10
-20
5
10
Retreat Longwall Extraction
Maximum Horizontal Stress Redistribution
magnitude of stressfield
15MPa
25MPa
Distance (m)
0
5
10
15
20
Retreat Longwall Direction
Vertical Stress Redistribution
Distance (m)
0
5
0
-5
5
Minimum Horizontal Stress Redistribution
CHANGE IN STRESS LEVELS
Relief
0 - -10MPa
-10 - -25MPa
Concentration
0 - 5MPa
5 - 10MPa
10 - 20MPa

**Fig. 9.4** Vertical and horizontal stress distribution about a longwall panel as determined from monitoring at a depth of around 500 m in the circumstances noted in the figure (After Gale and Matthews 1993; Gale 2014)

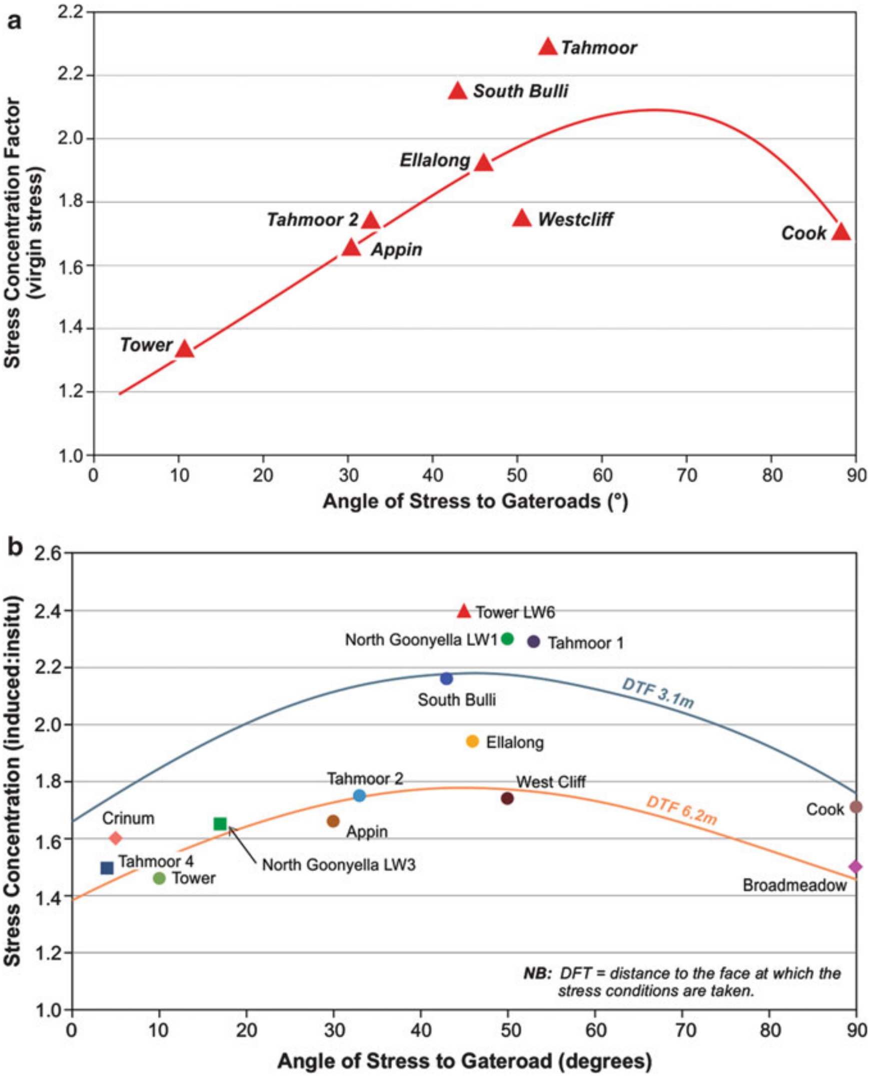


**Fig. 9.5** Relationships between orientation of gateroads relative to the major horizontal stress direction and horizontal stress concentration factor (After Gale and Matthews 1993; Gale 2014). (**a**) Relationship reported by Gale and Matthews (1993), (**b**) Updated relationship reported by Gale (2014)

are orientated within approximately 25° of the direction of the major horizontal stress. Stress relief is very well developed in stiff materials but not well developed in thick coal or soft materials (Gale 2014).

Stress notching tends to develop once the extraction face approaches to within about 30 m of an intersection but can be present over the full panel length and, if the stress is sufficiently high, extend into the companion gateroad (travelling road). In some instances, the intermediate/minor horizontal stress may also be of sufficient magnitude to result in stress notching. There is usually a marked increase in the impact of a stress notch when the extraction face is within 20 m of a maingate intersection, before the stress is relieved at the intersection. This stress relief is conducive to the unravelling of the strata fractured by the notching, resulting in intersection roof falls to a considerable height if pre-emptive support measures are not in place. Typically, these measures need to include long cables installed well ahead of the stress notch together with some form of surface support system. It is preferable to install this additional support prior to advancing the conveyor belt during gateroad development so that the placement of the support is not constrained at a later date by equipment, lack of access, or lack of space.

The floor is also subjected to the stress notching, which can result in floor heave making a substantial contribution to convergence. This may necessitate the installation of standing support, the leaving of additional coal in the floor, concreting of the floor and, in some cases, the bolting of the floor.

#### 9.2.5.3 Stage III – Travel Road/Tailgate Single Abutment

The significantly increased loading on the pillars and the presence of the adjacent goaf create the potential for a number of interactive behaviour modes to impact on pillar and roadway stability in single abutment loading situations. In the first instance, the increased pillar load results in compression of the pillar and its roof and floor strata. The strata adjacent to the goaf are free to dilate but this freedom progressively reduces with distance from the goaf edge back into solid abutment as self-confinement is restored. This generates an increase in horizontal stress in the immediate roof and floor strata due to the Poisson's effect. Depending on mining geometry, the Poisson's effect may be substantially recovered at the site of the travelling road (maingate companion road, which subsequently becomes the tailgate), thus subjecting the roof and floor strata of this roadway to elevated horizontal stress from this contributing factor.

If the increase in abutment stress is sufficiently large, it can initiate or aggravate yielding and crushing of the outer portions of the coal pillars. This results in an increase in the effective span of the travel road, thus reducing the resistance of both the roadway roof and floor to bending and buckling forces.

Crushing and yielding of the outer portions of a pillar give rise to a second source of induced horizontal stress in the roof and floor strata of the travel road/tailgate. The confined core concept for explaining pillar strength (Chap. 4) is premised on the outer crushed and yielding zones of a coal pillar providing confinement to the elastic core of the pillar (Fig. 4.22). In accordance with Newton's law of action and reaction, these restraining forces have to be balanced by compressive forces induced in the roof and floor strata. These compressive forces may induce the buckling and failure of the roof strata and/or the buckling and heaving of the floor beds (Salamon 1991) (Fig. 9.6).

In addition to the coal pillar element of the pillar system, consideration has to be given to the mechanical properties of the immediate roof and floor strata of the pillar system. Possible behaviour modes under the effect of high abutment stress include bearing capacity failure of the roof or floor strata and extrusion of soft roof or floor layers.

It has been suggested by some researchers that the immediate roof of a longwall travelling road/tailgate can be put into tension following the formation of the first adjacent longwall goaf. They attribute this to horizontal stress relief resulting from one or a combination of the presence of the goaf and differential pillar compression either side of the gateroad. The concept of horizontal stress relief due to the formation of a goaf is discussed in Sect. 5.2.5 and illustrated in Figs. 5.12, 5.13, and 5.14. The deviation of the in situ stress field around the zone of softening overlying a goaf may result in a reduction in lateral stress in the roof and floor of a travel/tailgate roadway but, in most cases, the residual component of lateral stress is still likely to be significant.

The concept that differential pillar compression associated with yielding coal pillars could contribute to the immediate roof of a tailgate being placed in tension (for example, as proposed by R. W. Seedsman 2012) appears to have its origins in stability concepts put forward by Diederichs and Kaiser (1999b) in relation to mining beneath blocky hanging walls in hard rock mines. The end-user is advised to carefully review the source publication to determine the relevance of the concept to their conditions.

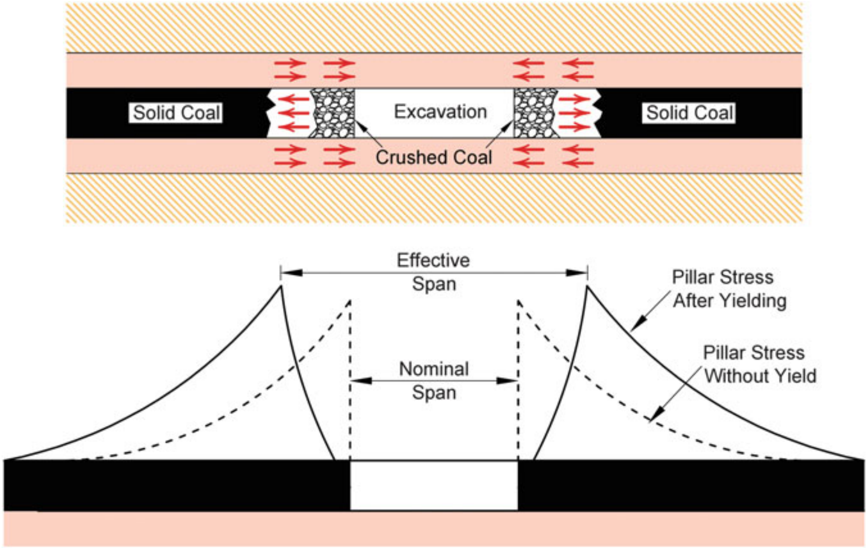


**Fig. 9.6** A schematic of pillar edge crushing showing the influence of the associated dilation and yielding on the lateral loading of the immediate roof and floor strata (Modified from Salamon 1991)

In some situations, roof displacement and floor heave can be associated with the failure and dilation of stiff, thin layers within the immediate roof or floor strata. These strata attract stress because of their stiffness before shearing and bulking so as to drive the roof down or the floor up, often in a rapid manner. This behaviour mode can be difficult to distinguish from pure buckling failure and from general bearing capacity failure. Two or more modes may be interactively in play at the same time.

Figure 9.7 shows an example of floor heave that developed dynamically in a tailgate companion roadway. The coal roof and pillar ribs were bolted and strapped. As the pillars dilated under high abutment load, the supported rib line remained intact but started to ride over the W straps and rock bolts installed in the roof. Ultimately and without warning, the coal floor uplifted. Vasundhara et al. (2003) provide more detailed discussion on weak floor failure mechanisms associated with longwall mining operations.

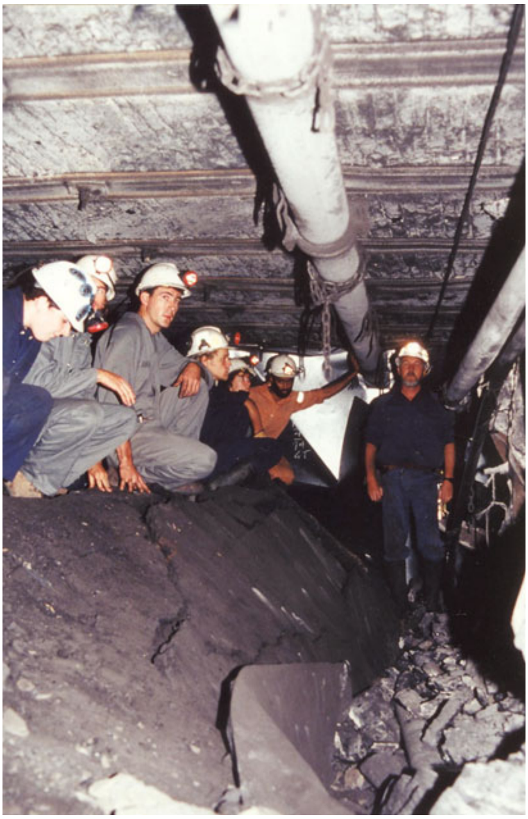


**Fig. 9.7** Dynamic heave of a coal floor beam in a gateroad located adjacent to a highly loaded chain pillar

A range of operational benefits is associated with not installing standing support in a longwall travelling road/tailgate. These relate to ventilation efficiency, inspections, material and equipment access, and labour requirements. Hence, there has been a focus on replacing standing support systems with long tendons. This has met with success at some mines. However, standing support continues to be required in those mining environments where floor uplift constitutes a significant component of seam convergence and where the effectiveness of tendon support systems is adversely affected by shear displacement on bedding planes. The timing of the installation of standing support is a matter for site management, as dictated by local conditions and mining priorities. However, when standing support is required, it is strongly advisable to always have it in position for 100 m outbye of the longwall face and, preferably, for at least 200 m.

#### 9.2.5.4 Stage IV – Tailgate/face

Ground behaviour in the vicinity of the tailgate end is complicated further by two factors. Firstly, the chain pillars are subjected to additional increases in abutment stress. Secondly, the immediate and upper roof strata have another degree of freedom, with the opportunity to displace both transversely into the existing adjacent goaf and longitudinally into the approaching new goaf. Weak bedding planes in the roof facilitate large scale slip of the roof strata towards the goaf, as measured for example by Fabjancyk et al. (2006). Assessment of strata deformation modes and impacts in these environments falls outside the scope of empirical and semi-empirical approaches to pillar stability assessment.

One type of behaviour specific to this environment is the so-called **skew roof mechanism,** which Tarrant (2005a) and Fabjancyk et al. (2006) associate with a change in the profile of a tailgate from rectangular to rhomboidal as shown in Fig. 9.8. The behaviour, which can vary in magnitude and direction between mine sites, has been attributed to the reorientation of the stress field around the goaf generating shear couples on bedding planes and other structures in the roof and floor. These shear couples result in differential shear within the strata, leading to high levels of strata failure in the roadway. According to Fabjancyk et al. (2006), once a skew roof mechanism is initiated, it is likely to extend a substantial distance into the goaf. The magnitude and direction of pre-mining horizontal stress is believed to have a major impact on the direction of the skew and the extent that the skew process impacts on the roadway. Tarrant (2005a) lists the key factors driving skew roof behaviour as:

- the absolute and relative magnitudes of the vertical and horizontal stresses;
- the shear modulus of the strata pile (shear deformability); and
- the extent of overburden bridging.

Tarrant credits shear stress damage due to skew roof behaviour with being capable of destroying intrinsic support, including cable bolts. Hence, standing support rather than cables is considered the most appropriate stabilisation strategy, with Tarrant (2005b) providing a range of recommendations in that regard.

Fabjancyk et al. (2006) provide further discussion of the skew roof mechanism, concluding that the range of strata deformation mechanisms that can occur around goaves warrants that the positioning of roadways in the vicinity of goaves is based on a higher level of assessment than that used for traditional pillar stability approaches. Moodie and Anderson (2011) report on similar behaviour associated with longwall top coal caving at Austar Coal Mine, Australia, where change in vertical stress was measured to be higher on the travel road side of a chain pillar rather than on the goaf side.

In summary, pillar and roadway behaviour about a tailgate can be complex. It can involve a range of stress paths and mechanisms. All may have application in some situations but none are exclusive. Different mechanisms and combinations of mechanisms operate in different environments and at different points in time in the mining process. As already noted in Sect. 5.2.5, each situation should be individually assessed, with consideration given to utilising

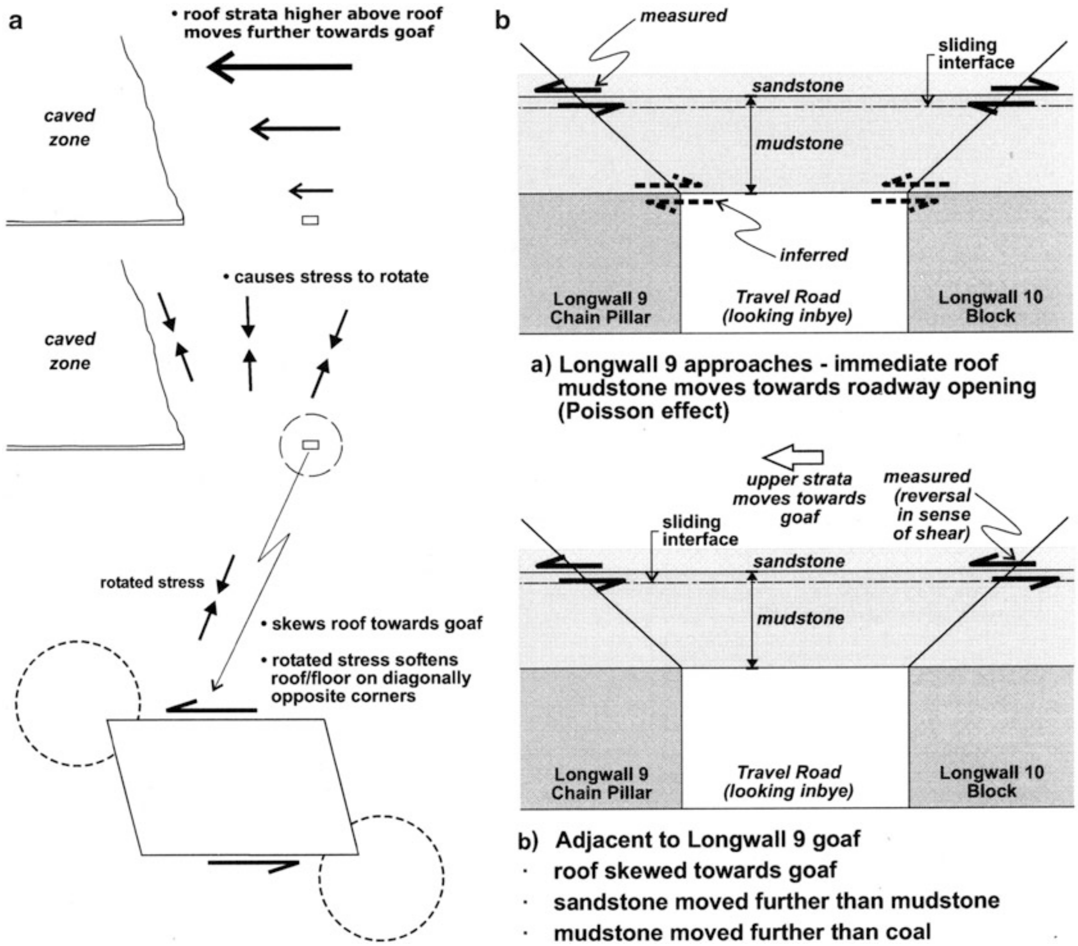

a
• roof strata higher above roof moves further towards goaf
caved zone
• causes stress to rotate
caved zone
rotated stress
• skews roof towards goaf
• rotated stress softens roof/floor on diagonally opposite corners
b
measured
sliding interface
sandstone
mudstone
inferred
Longwall 9 Chain Pillar
Travel Road (looking inbye)
Longwall 10 Block
a) Longwall 9 approaches - immediate roof mudstone moves towards roadway opening (Poisson effect)
upper strata moves towards goaf
measured (reversal in sense of shear)
sliding interface
sandstone
mudstone
Longwall 9 Chain Pillar
Travel Road (looking inbye)
Longwall 10 Block
b) Adjacent to Longwall 9 goaf
· roof skewed towards goaf
· sandstone moved further than mudstone
· mudstone moved further than coal

**Fig. 9.8** Concepts developed by Tarrant (2005a) relating to the skew roof mechanism in longwall tailgates. (**a**) Simplified model of stress/displacement changes adjacent to goaf. (**b**) Relative movement as monitored at Metropolitan Colliery, Australia

numerical modelling to predict principal stress magnitudes and profiles and to give insight into strata behaviour modes.

#### 9.2.5.5 Stage V – Double Abutment Loading

Full double abutment loading situations are not usually of interest other than if the chain pillars have a role to play in restricting surface subsidence or if multiseam mining is contemplated. In the case of surface subsidence, the structural integrity of the coal pillar and the compression of the chain pillars and surrounding strata take on added significance. It should be noted that surface subsidence above a chain pillar is not necessarily an indicator of the state of stability of the pillar. This is because elastic strata compression, especially at depth, can make a major contribution to surface subsidence.

Particular care is required when using surface subsidence behaviour above chain pillars to draw conclusions about their state of stability. Some vertical surface displacement will occur over any chain pillar simply due to elastic compression of the coal pillar and surrounding strata in response to mining-induced stress. This compression can be quite considerable at depth. Lateral displacement of the overburden towards the goaf is another factor that contributes to the development of vertical displacement above chain pillars.

## 9.3 Longwall Powered Supports

### 9.3.1 Development

Longwall mining of coal originated in the 1800s as a so-called ‘hand-got’ mining method (pick and shovel) using timber props with headboards as face support. Hand-got mining was progressively replaced with shotfiring, ploughs and shearers. Support progressed to rows of friction props with connecting bars that were leapfrogged forward with face advance. Hydraulic props were introduced in the 1940s, followed by the Eastern European concept of a shield support comprised of a rigid, half-arch, frame to protect the face from goaf flushing. These early support systems provided the basis for the first longwall **powered supports**, so-called because they were connected to a hydraulic power supply and capable of self advancing.

In Russia, the rigid shield was developed into a hydraulic **shield support** by pinning a canopy to the flushing shield and connecting the flushing shield to a base by a simple hinge and one or two hydraulic legs (Figs. 9.9a and 9.10). These supports were often referred to as **caliper** or **arc shields** since the canopy tip followed a circular pathway as the support was raised or lowered. The concept was developed further in Germany in the 1960s, with these shield supports being characterised by rear-facing angled legs; a relatively high tip load capacity; a low rear load carrying capacity; good protection against goaf flushing; and a high longitudinal stiffness to resist horizontal displacement towards the goaf. Because the legs of a shield support are angled, the vertical support provided to the roof is less than the rated capacities of the legs and reduces as the legs become more inclined when the support yields.

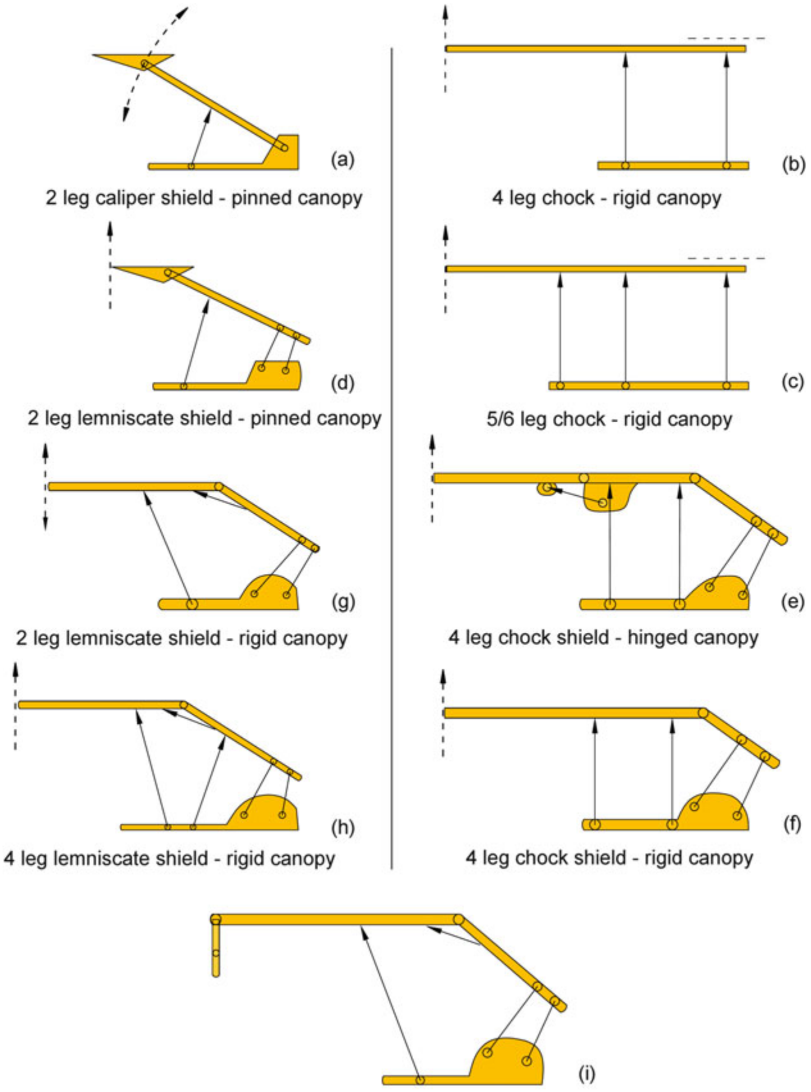
The first hydraulic supports in Britain were installed in 1951. These comprised hydraulic legs mounted on a base, with connecting bars replaced with a solid canopy to produce a so-called hydraulic **chock support** (Fig. 9.9b). The British National Coal Board (NCB) stipulated a maximum distance between the coal face and the front support leg of 2 m, which effectively precluded the use of 2 leg shield supports. The NCB also dictated that the full face height had to be cut in one pass (bi-directional shearing) and that the face was operated in so-called **conventional mode**, whereby the supports were set up against the armoured face conveyor (AFC) prior to the passage of the shearer and advanced immediately after its passage.

The British powered supports were characterised by four or more vertical hydraulic legs, a high rear load carrying capacity, a low tip load carrying capacity, poor protection from flushing of the goaf, and poor longitudinal stability, the latter making them prone to collapse in a scissor-like manner as the immediate roof strata moved towards the goaf. In order to address the low tip load capacity and to satisfy NCB specifications, additional hydraulic legs were fitted to the front of some supports (Figs. 9.9c and 9.11).

A major design development in longwall hydraulic supports occurred when the German coal industry replaced the simple hinge on the shield support with a **lemniscate linkage**. The lemniscate linkage caused the canopy tip to travel in a near vertical plane as the support was raised and lowered and imparted high longitudinal stability to the support (Fig. 9.9d). This addressed the concern that the arc motion of a caliper shield resulted in an unfavourable reduction in confinement to the immediate roof when the support was set and a favourable increase in confinement as the support yielded and converged. Subsequently, the concept was incorporated into chock supports to prevent them from collapsing into the goaf, thereby giving rise to the **chock shield** (Fig. 9.9e).

In the meantime, shield supports were fitted with a rigid canopy instead of a pinned canopy and forward angled legs became the norm, with both features increasing tip load capacity (Figs. 9.9g and 9.12). In the 1970s, in response

**Fig. 9.9** Chronology of the development of longwall powered supports



2 leg caliper shield - pinned canopy

4 leg chock - rigid canopy

2 leg lemniscate shield - pinned canopy

5/6 leg chock - rigid canopy

2 leg lemniscate shield - rigid canopy

4 leg chock shield - hinged canopy

4 leg lemniscate shield - rigid canopy

4 leg chock shield - rigid canopy

(i) High capacity 2 leg shield with extended rigid canopy and flipper capability

primarily to a marketing ploy by some chock shield manufacturers that supports with four legs 'obviously' had to offer support benefits over shields with only two legs, an additional pair of legs began to be incorporated into shield supports (Fig. 9.9h). These supports were short-lived, with engineering analysis by McKay (1978) and others of the heavier, more complex and more costly 4 leg supports, concluding that the additional legs offered little, if any, additional support benefits and, in some instances, reduced performance.

Up until the late 1970s, many chock shields employed a hinged canopy, also referred to as a split or cantilevered canopy. The location of the hinge point ranged from between the front and back legs to well in front of the front legs, although the hinge point immediately in front of the front legs was most common (Figs. 9.9e and 9.13). This allowed the canopy to adapt better to

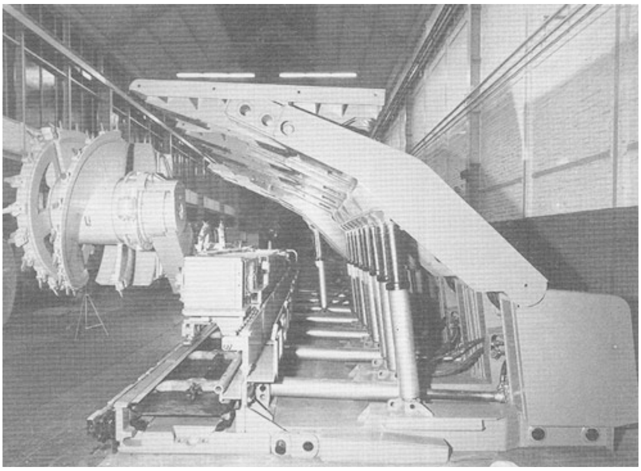


**Fig. 9.10** 2 leg caliper shield powered supports, shearer and AFC for the longwall face reported by Cloete (1980) to have set a world record of a monthly production of 162,557 tonnes at Sigma Colliery, South Africa, in June 1980

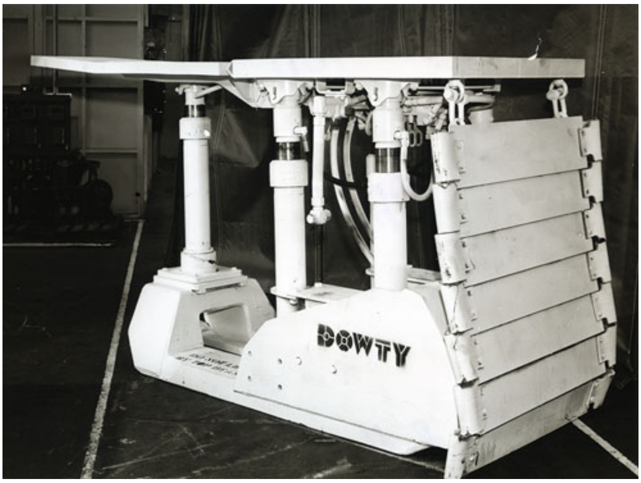


**Fig. 9.11** A five leg, cantilevered canopy, chock powered support

the shape of the roof, which often contained vertical steps because the shearer cutting horizon was controlled manually. The tip load capacity of these supports was independent of the leg capacity, being determined by the cantilever cylinder capacity and its lever arm distance. The supports generated low tip loads, typically one-tenth of the nominal support capacity and, with the onset of yielding, the cantilever extensions were prone to a domino collapse along the face. A failure of this type involving supports with a tip capacity of only 100 kN (10 t) occurred on the first longwall face at Coalbrook Colliery, South Africa, in 1979 (Henderson 1980; personal experience). These types of incidents contributed to cantilevered canopies being phased out in favour of rigid canopies (Fig. 9.14).

Major advances in extrusion technologies in the 1990s enabled hydraulic leg capacity to be more than doubled, from typically 2–5 MN (~200 to 500 t). Leg capacity has continued to increase, approaching 0.9 MN (900 t) by 2010, with corresponding increases in tip load capacity and in leg stiffness due to the larger bore area. The configuration and high tip load capacity of shield supports has enabled the length of the rigid canopy section to be extended while maintaining a very high tip load capacity, so that longwall faces are now operated routinely with the powered supports set back from the AFC a

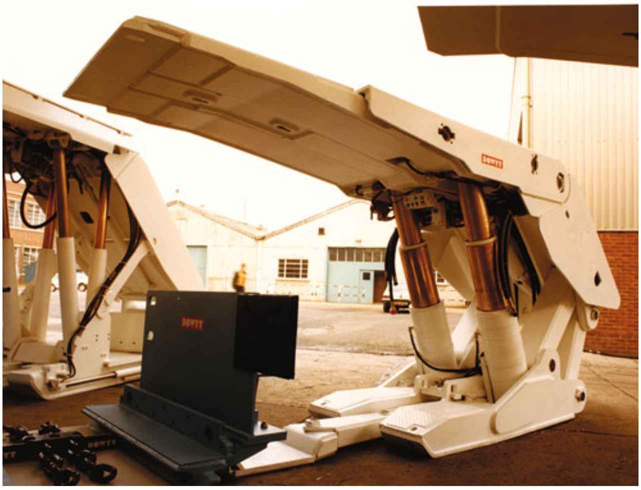


**Fig. 9.12** A 2 leg, rigid canopy, shield powered support fitted with a lemniscate linkage

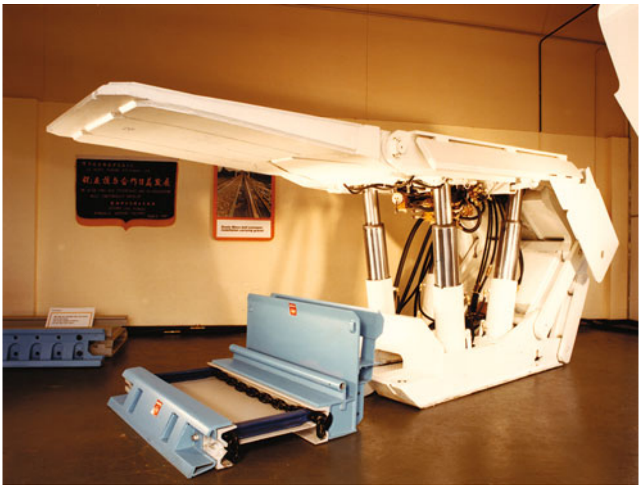


**Fig. 9.13** A 4 leg, cantilevered canopy, chock shield powered support

distance of one cutting web. The additional space gained in this so-called **one-web back** mode provides a number of operational benefits, such as improved ventilation, larger face conveyors, and a second travel (walk) way along the face. In the event that face conditions deteriorate and/or the face spalls excessively, the option still exists to 'close up the face' by advancing the supports a distance of up to one web, although care is then required when taking the next shear to avoid cutting into the support canopies. Closing up the face in this manner is referred to as **double chocking.**

The substantial improvements that have been achieved in tip support capacity and minimisation of the area unsupported between support tips and the face can be negated if the coal face spalls, especially in thicker seams. To address this problem, it is now very common for powered supports utilised in thicker coal seams to be fitted with an hydraulically activated extension, or **flipper**, that can be deployed as either or both an extension to the canopy to confine the immediate roof and a face sprag to confine the coal face (Figs. 9.9i and 9.15).

In order to accommodate larger diameter legs and longer canopies and flippers and to improve the lateral and torsional stability of shield supports in thick seam mining operations, the width of powered supports has progressively

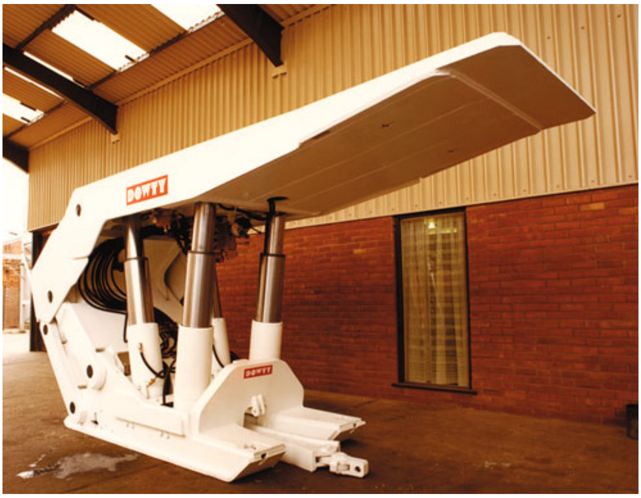


**Fig. 9.14** A 4 leg, rigid canopy, chock shield powered support

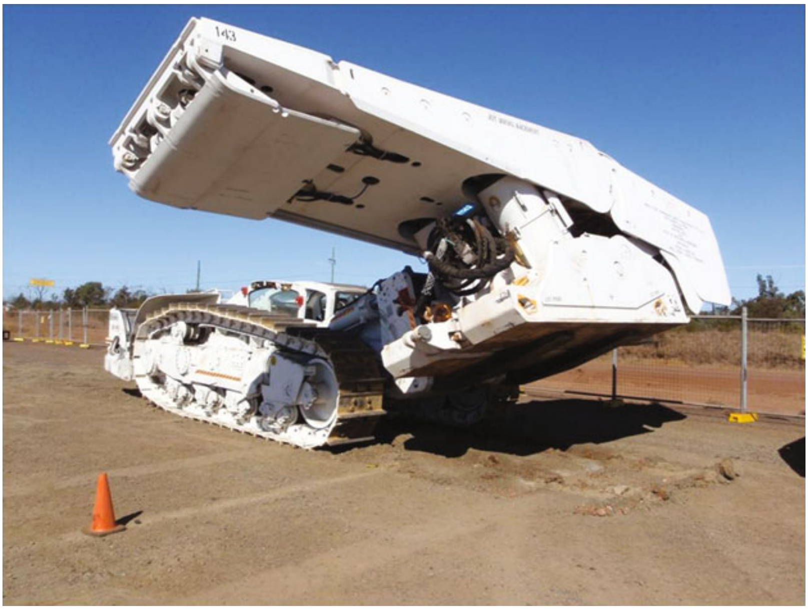


**Fig. 9.15** A 2 leg, 17.5 MN capacity, shield powered support fitted with an articulated flipper

increased from the traditional 1.5 m to upwards of 2.0 m. Hence, shield supports now offer advantages over chock shield supports in terms of higher tip load capacity; increased vertical stiffness; reduced number of components; less structural complexity; and reduced size and reduced weight, while at least matching the rear support capacity of chock shields. Shield supports are standard on all new longwall faces in Australia and the USA. However, 4 leg chock shields continue to be utilised in longwall top coal caving operations in thick seams.

### 9.3.2 Basic Functions

Effectively, powered supports are located in the goaf and, therefore, are surrounded by strata that have already been impacted by mining-induced fracturing. The basic ground control functions of a powered support are to maintain this fractured strata in a confined and interlocked state; control convergence in the face area to limit further localised fracturing and bedding plane movement; and provide a goaf break off line. These functions are not mutually exclusive.

Excessive convergence generates additional fracturing and leads to increased rib spall and guttering at the face, and bed separation and block detachment above the supports, all of which are conducive to roof falls on the longwall face. Face spall increases the unsupported distance between the tip of the longwall supports and the face. Guttering results in roof cavities and, together with bed separation and block detachment, increases the load acting on the supports. In turn, this increases the likelihood and magnitude of support yield, resulting in more convergence. The process can become self perpetuating as yielding of the powered supports results in increased face load, bed separation, and fracturing. Failure to induce caving of the immediate roof at the rear of the powered supports aggravates these conditions.

In addition to providing support to the roof, a powered support assists in sustaining horizontal stress in the immediate roof strata to confine the fractured rock and maintain it in an interlocked state so that it does not unravel on the face line. This is accomplished by sandwiching the immediate roof between the support canopy and the upper strata, thereby maintaining bedding planes in a clamped state to resist horizontal displacement and dilation as the strata subsides onto the goaf pile. This function requires the support to have the capacity to transfer horizontal thrust to the floor, which is achieved through the lemniscate linkages.

### 9.3.3 Static and Kinematic Characteristics

The performance of a powered support is dependent on its static and kinematic characteristics. Two conditions must be satisfied for a powered support to be in a state of equilibrium, namely, the algebraic sum of all forces acting on it must be zero, and the algebraic sum of all moments about any point must be zero. Other parameters of particular importance to shield performance are:

- total roof support resistance of the support;
- support resistance of each load bearing member;
- stiffness of the support;
- canopy ratio (or canopy balance, being as discussed later, the ratio of canopy face tip to leg distance to canopy rear end to leg distance);
- capacity to vary canopy attitude;
- immediate roof and floor bearing pressure and capacity; and
- the kinematic properties of the support for adapting to various roof geometries.

The computation of load acting on a longwall powered support is complex and statically indeterminate. It is a function of the stiffness of the powered support and the stiffness of the surrounding strata, both of which can vary during the mining process and be time dependent. There are numerous permutations in the factors that determine the system stiffness. These include:

- depth of mining;
- mining height;
- composition, thickness and caveability of the immediate roof strata;
- composition, thickness and caveability of the upper roof strata;
- relative location and thickness of particularly weak, strong or extrusive strata;
- strength of the floor strata;
- joint direction, dip and density;
- density of mining-induced fracturing;
- configuration of the powered support;
- stiffness of the powered support; and,
- setting and yield pressure of the powered support.

Hence, no geotechnical model finds universal application and each site has to be assessed in its own right using tools such as surface to seam displacement instrumentation; microseismic monitoring; powered support pressure and convergence monitoring; surface subsidence monitoring; numerical modelling; and observation

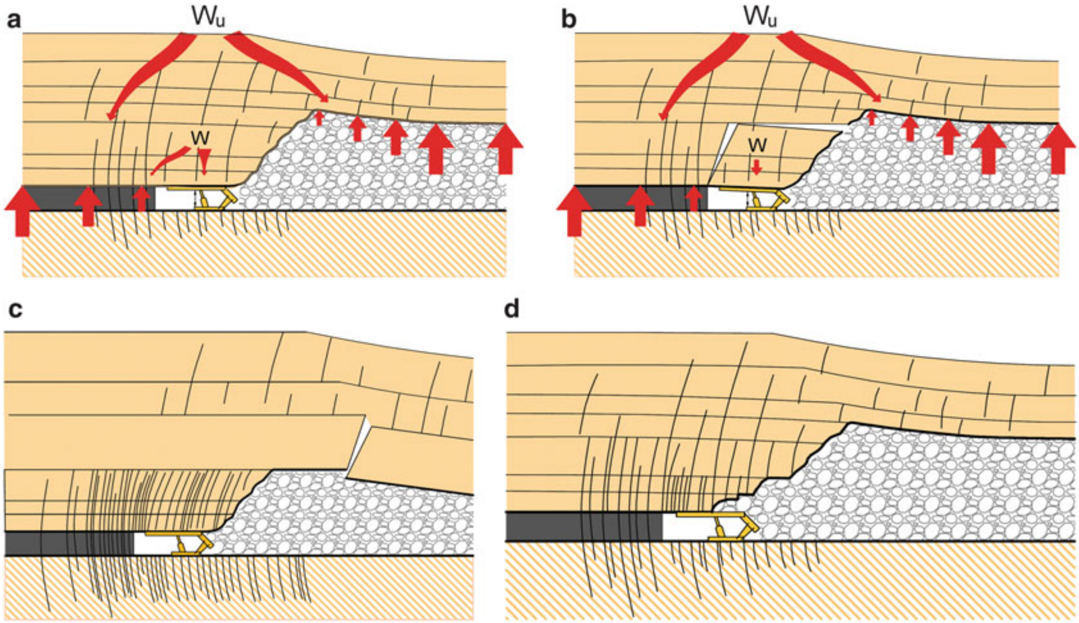


**Fig. 9.16** Conceptual loading models for longwall powered supports. (**a**) Bulking model, (**b**) Detached block model, (**c**) Periodic weighting model, (**d**) Unconfined model

and deduction. The four models depicted in Fig. 9.16 provide a basis for conceptualising powered support statics across the range of conditions most commonly encountered.

There is no unique definition of **total roof support resistance**, also referred to as **total support density**, but it is most commonly defined as the ratio of the total normal thrust applied by the support to the roof, to the area of roof supported by each powered support unit. This area is measured from the coal face to the edge of the last supporting element on the goaf side of the face. The total thrust is based on the sum of the nominal yield loads of all the hydraulic support elements in the system. The support resistance is a minimum immediately after the passage of the shearer and a maximum once the support has been advanced.

Early developments in determining total support resistance were based around British views that support resistance need only be sufficient to prevent excessive convergence and European views that it had to be as high as possible to prevent bed-separation over the face area. Many of the European views related support resistance to mining height, reasoning that the greater the mining height, the greater the caving height and, therefore the thicker the strata bed resting on the longwall face supports. At the time, minimum support resistances on installed faces ranged from 100 kN/m$^2$ to 1.2 MN/m$^2$ (10–120 t/m$^2$), the latter associated with strong massive roof strata situations. With the benefit of hindsight, it appears that the difference between the two philosophies was simply a reflection that typical British strata behaved in a more plastic manner and, therefore, was more tolerant of convergence than the more massive and brittle strata associated with European conditions.

Since the early 1990s, it has become common practice in weak to moderately strong roof strata in Australia and the USA to operate powered supports at a set pressure of 0.6–0.8 MN/m$^2$ (60–80 t/m$^2$) and a yield pressure of 1–1.1 MN/m$^2$ (100–110 t/m$^2$). Some operations use a higher set pressure of 90 % of yield pressure. These appear to be limiting values when the contact strength of the roof strata is taken into account and to be supported by numerical modelling outcomes. Gale (2009), for example,

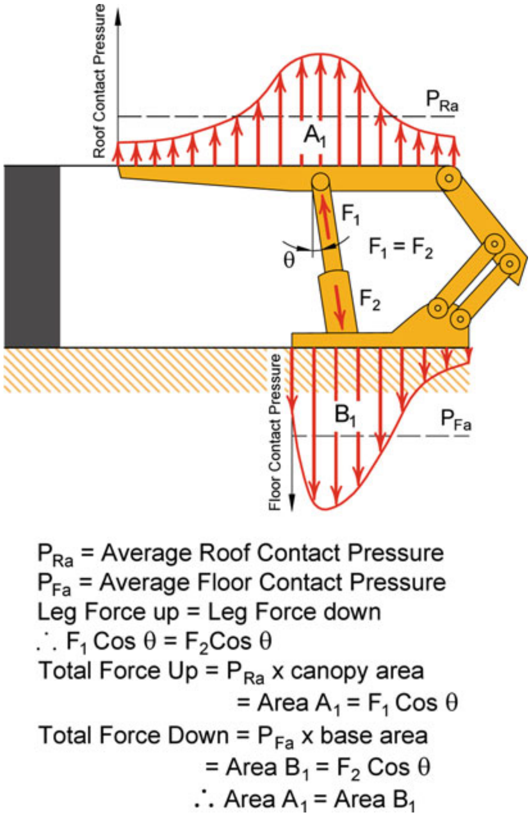

Roof Contact Pressure
$P_{Ra}$
$A_1$
$F_1$
$\theta$
$F_1 = F_2$
$F_2$
Floor Contact Pressure
$B_1$
$P_{Fa}$
$P_{Ra}$ = Average Roof Contact Pressure
$P_{Fa}$ = Average Floor Contact Pressure
Leg Force up = Leg Force down
$\therefore F_1 \cos\theta = F_2 \cos\theta$
Total Force Up = $P_{Ra}$ x canopy area
= Area $A_1 = F_1 \cos\theta$
Total Force Down = $P_{Fa}$ x base area
= Area $B_1 = F_2 \cos\theta$
$\therefore$ Area $A_1$ = Area $B_1$

**Fig. 9.17** Idealised distribution of roof and floor contact pressure about a powered support

has concluded on the basis of numerical modelling that support resistances over 1.2 $MN/m^2$ (120 $t/m^2$) would be considered excessive and not required in weak environments. In stronger and more massive roof strata, higher values for support resistance, setting and yield pressure prevail. As at 2010, the highest capacity shield supports in the world had a total support resistance of 1.6 $MN/m^2$ (160 $t/m^2$) (Winter et al. 2010). These were employed at Moranbah North Mine, Australia, beneath a weak immediate roof overlain by a strong massive roof prone to periodic weighting.

It is important to appreciate that the total support resistance is not uniformly distributed over the roof and the values quoted earlier and those provided in manufacturer's specification sheets are averaged over the full roof area. In the ideal case of the powered support being sandwiched between two rigid plates, the maximum support resistance is generated at the end points of the hydraulic legs, as illustrated in Fig. 9.17. Load transfer to the roof reduces with distance along the canopy from the legs. A similar load transfer profile exists in the floor. This situation approximates to that associated with the bulking model depicted in Fig. 9.16a.

The bulking model can be conceptualised as a displacement controlled system, with irresistible strata convergence of the upper roof strata loading the coal face, the powered supports and the goaf. The powered supports represent very soft springs located between stiff springs, being the adjacent goaf, and very stiff springs, being the coal face. The stiffness of the legs of the powered supports and their setting load determine the overall stiffness of the powered support and, therefore, the amount of convergence that can occur prior to the supports reaching yield. In this setting, powered supports only have the capacity to influence strata behaviour in their immediate vicinity. The concept of 'bigger is better', in terms of support resistance, does not necessarily deliver improved face control. Rather, the more critical controlling factors may be the point of application of support resistance; load distribution within the canopy and the base; support stiffness; the integrity of the immediate roof to function as a fractured but interlocked beam or cantilever; and roof and floor contact strengths.

The behaviour of the bulking model is changed significantly if a face break occurs, resulting in a detached block above the powered support (Fig. 9.16b). The detached block causes the system to revert from being displacement controlled to being load controlled. Ashwin et al. (1970), Whittaker (1974) and Wilson (1975) proposed similar simple analytical models for determining the distribution of forces and moments for this situation. While there are a number of limitations associated with these models (see for example, Smart et al. 1982; Aziz and Porter 1985) which have resulted in modifications by Smart and Redfern (1986), Barczak and Tadolini (2007), and others, they

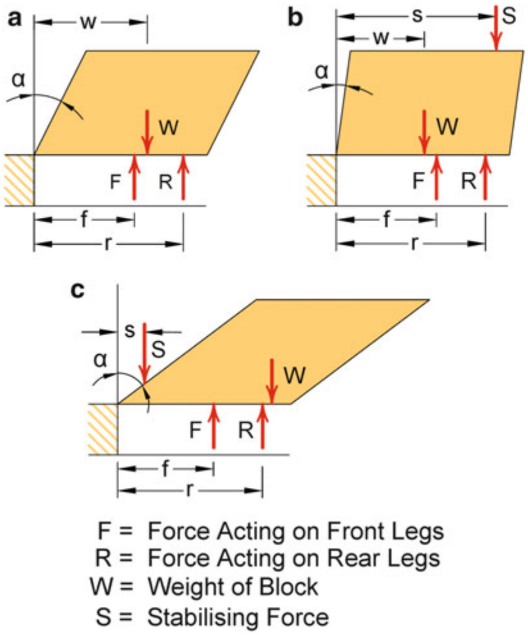


**Fig. 9.18** Detached block model geometries for a 4 leg rigid canopy chock shield powered support

still give valuable insight into the basic behaviour of powered supports and provide a reasonably accurate analysis of one extreme condition.

The detached block models assume that the legs of the powered support are rigid and that the support is uniformly loaded by the dead weight of the detached block of strata of mass, W. The block can be of any size and shape, although some models define its geometry on the basis of caving height, caving angle and overhang distance into the goaf. Geometry determines where the centre of gravity, or centroid, of the detached block acts on the powered support. This may be on the face side of the legs, the goaf side of the legs, directly over the legs of a shield support, or between the front and back legs of a chock shield. In order to prevent rotation, a fictitious balancing or stabilising force, S, has to be introduced to mimic the resistance to rotation provided by the roof strata (Fig. 9.18).

For the case where the centre of gravity acts between the face and the front legs of a rigid canopy chock shield (Fig. 9.18a), the maximum weight, W, of loosened strata that can be supported is found by taking moments about the rear of the support and is given by Eq. 9.2.

$$W = F\left(\frac{s-f}{s-w}\right) \tag{9.2}$$

where

*F* = *normal component of combined capacity of front legs*
*B* = *normal component of combined capacity of rear legs*
*s, w, f and r* = *lever arm distances*

Assuming that the yield ratings of the front and back legs of the chock shield are equal, it follows from Equation 9.2 that because (s-w) is greater than (s-f), the actual support resistance is less than one half of the nominal support resistance. A similar static analysis can be performed for 2 leg shields and for 4 leg chock shields when the centre of gravity of the load acts between the two sets of legs or behind the rear set of legs. The periodic weighting model (Fig. 9.16c) represents an extreme case of the detached model in which the centre of gravity of the load acting on the support is some distance back into the goaf.

The model demonstrates that the total thrust of a support system is only ever equal to the sum of the nominal thrust of the system components when the centre of gravity of the load acts directly over the legs on a 2 leg shield support or at the mid-point between the front and back legs on a four leg chock shield. Longwall support manufacturers utilise the detached block model to compute and specify the tip and rear load capacities of powered supports, examples of which are presented in Table 9.1. This table shows that when the centre of gravity of the supported load acts at the tip, the actual load carrying capacity of the powered support is of the order of only 25 % of the its nominal support capacity.

The detached model highlights the importance of considering not only total support

**Table 9.1** A selection of manufacturer's specifications for longwall powered supports

| Support type | Total leg support capacity (MN) | Pre-cut support resistance (MN/m$^2$) | Maximum support capacity when centre of gravity acts at: Tip (MN) | Rear (MN) | Average roof bearing pressure at yield (MN/m$^2$) | Average floor bearing pressure at yield (MN/m$^2$) |
|---|---|---|---|---|---|---|
| 4 leg chock shield | 8.0 | 0.77 | 1.91 | 6.07 | 1.17 | 3.13 |
| 4 leg chock shield | 9.0 | 0.87 | 2.13 | 6.83 | 1.26 | 2.35 |
| 2 leg shield | 9.8 | 1.05 | 2.56 | 7.29 | 1.30 | 3.59 |
| 2 leg shield | 12 | 1.30 | 3.49 | 8.56 | 1.45 | 2.68 |
| 2 leg shield | 17.48 | 1.50 | 5.06 | 12.42 | 1.60 | 3.20 |

resistance when selecting powered supports but also the location and distribution of turning moments that may be generated within the support. However, the model has limitations, as becomes evident when it is applied to high capacity shield supports. The model cannot cause the supports to yield under any realistic detached block configuration other than one which cantilevers at least 10–15 m into the goaf, such as encountered in some periodic weighting situations. The model is unable to account in its own right for other situations in which shield supports yield. This partially reflects the fact that powered supports do not have the capacity to resist all mining-induced convergence, with the level of convergence required to cause yield decreasing with increase in powered support stiffness associated with higher set pressures and stiffer hydraulic legs.

The unconfined model represents the situation where the caving line progresses over the top of a powered support (Fig. 9.16d). This is more likely to occur at larger mining heights in weak strata environments. It results in a relaxation in lateral confining stress at the face, allowing the fractured strata between the tip of the support and the face to unravel. Factors which aggravate the situation include the presence in the immediate roof of low friction bands and bands prone to extrude under load; an irregular roof cutting profile; the presence of a cavity associated with a previous face fall; sloppy lemniscate linkages; and inadequate support resistance.

The progression of the caving line towards the face increases the turning moments at the tip of the support because it simultaneously removes counter balance from the rear of the canopy and moves the centre of gravity of the load towards the tip. Once the cave line reaches the front legs, the canopy is free to rotate about these legs, allowing the tip to drop into the working place and reducing tip capacity to zero. Face falls are inevitable without intervention to fill voids and reconsolidate the fractured strata.

The load distribution profile, maximum tip capacity, and maximum rear capacity of a shield support are very sensitive to the **canopy ratio**, or **canopy balance**, defined by Eq. 9.3 as:

$$\text{Canopy Ratio, or Canopy Balance} = \frac{\text{Distance from tip to legs}}{\text{Distance from legs to rear}} \qquad (9.3)$$

A misconception sometimes associated with a shield support is that angling of the legs towards the face introduces a horizontal component of stress to confine the immediate roof, with this confinement increasing as the support yields.

This is not the case as the lemniscate linkage causes the support canopy to travel in a straight vertical trajectory (Fig. 9.9).

In addition to maintaining forces and moments in equilibrium, the capacity of a support to control convergence depends on the stiffness of its hydraulic system and on its setting and yield loads. Hydraulic system stiffness is determined primarily by the height and area of the fluid column in the legs, with a component also associated with expansion of the leg tubes and hoses. In accordance with Eq. 2.3, everything else remaining unchanged, the higher the fluid column in the legs, the less pressure (or support resistance) developed per unit of convergence. The setting load corresponds to a prestress applied to resist convergence, while the yield load determines the peak resistance to convergence. Although longwall mining height has increased substantially and now approaches 6 m, the corresponding reduction in leg stiffness has been offset to some degree by the larger bore diameter of the hydraulic legs associated with modern thick seam supports. In the case of double telescopic legs, the load generated by the support is determined by the cross-sectional area of the smallest cylinder in the telescopic leg.

Care has to be exercised in relying on some stiffness values and concepts for longwall supports presented in the literature as there is a mix of definitions of stiffness, some computations are flawed, and some concepts are confused. Typically, a load increment of 1 MN (100 t) with its centre of gravity acting in the thrust line of the legs of a modern 2 leg shield support extended to 3 m will result in 5–7 mm of convergence up to the yield point of the support, corresponding to a support stiffness of 0.14–0.2 MN/mm.

However, a lower load is required to produce the same convergence if the centre of gravity of the load acts in front of or behind the thrust line of the legs, or if the support operates at a greater height. If the effective area supported by a 2 leg shield is approximated to be 10 $m^2$, then a 1 MN (100 t) load increment acting over the same shape and size area on a 3 m high coal face would result in around only 0.6 mm of convergence. Hence, the effective stiffness of a powered support is an order of magnitude less than that of the coal that it replaces, meaning that even in the most favourable circumstances, a powered support only makes a small contribution to controlling the overall stress and convergence distribution about a longwall face.

If debris accumulates over or under a powered support, it acts as a soft inclusion and can negate the benefit of high leg stiffness to control convergence. Good housekeeping to minimise the accumulation of this material, high setting pressures, and maintenance of setting pressures to compact the material are important in minimising convergence. Skimming the roof with the canopy of a powered support as it is advanced also assists in minimising debris on top of the canopy.

In specifying the support resistance for a longwall powered support, careful consideration needs to be given to the contact strength and bearing capacity of the immediate roof and floor strata and to the loading profile of the support canopy and base. Contact pressures are higher at the floor than at the roof due to the smaller load bearing area of the support base and the effect of turning moments (Fig. 9.17). The combination of leg configuration and high tip load capacity of a shield support can generate high turning moments and, therefore, concentrate loadings at the toe of these types of supports. Hence, the bearing capacity of the floor is an important consideration when designing a powered support for a specific site or assessing if a powered support is suitable to a different site. It has a significant influence on powered support design in respect of:

- the overall geometry of the powered support and AFC so that base loading profiles do not exceed the bearing capacity of the floor;
- the type of base fitted to the support (solid or split); and
- the fabrication of the base to tolerate bending and torsion over its planned operating life.

A range of approaches can be adopted to assessing the bearing capacity of the floor, with the most common being the application of bearing capacity formulae of the type presented in Appendix 4 and numerical modelling. Solid bases to maximise load carrying area and base lifting rams to raise the front (toes) of the powered supports when advancing them are two controls utilised in weak floor strata to mitigate against bearing capacity failure. If the bearing capacity of the floor is exceeded, powered supports start to rotate towards the face, resulting in the unloading of the canopy at its tip. In these situations, shield supports are prone to topple towards the face, especially once mining height exceeds about 3 m. On the other hand, uneven, hard floor conditions can subject the base of a support to excessive bending and torsion, leading to the failure of welds. Split base support systems offer some advantages in these conditions because torsional forces on the base are greatly reduced.

The computation of load and turning moments acting on a longwall support is complicated because in addition to being statically indeterminate, it is also time dependent. Medhurst (2005) proposed that the ground response curve concept provided a convenient means to graphically show ground behaviour, its relationship to powered support performance, and roof stability. The basis of this approach is shown in Fig. 9.19. It is premised on roof behaviour being convergence controlled, with a unique ground response curve applying to each combination of mining conditions (geology, depth, geometry etc.).

In practice, considerable uncertainty is associated with the calculation of a ground response curve for a longwall face environment because of the numerous complex permutations of strata behaviour about a longwall face, their time dependency, and a lack of data over the full range of a ground response curve. Medhurst (2005) proposed that a strata-support interaction relationship of the type defined by the curve AD in Fig. 9.19 could be derived by considering:

- routine geotechnical data;
- leg convergence/stiffness test results;
- monitoring data relating to leg pressures, surface subsidence, surface to seam extensometers and microseismics; and
- underground observations.

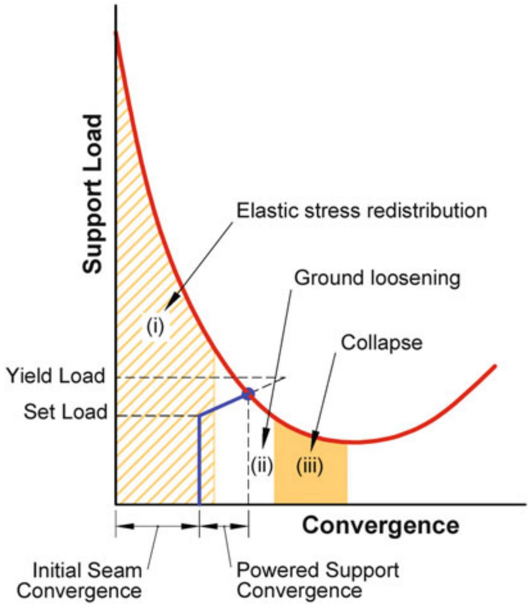


**Fig. 9.19** Ground support interaction curve applied to a longwall face (Adapted from Medhurst (2005) and Gale (2009))

A limitation with this approach, as recognised by Barczak (2006) and others, is that the mine loading conditions are not sufficiently comprehensive and controlled to develop a full curve. Numerical modelling can assist but it is still constrained by the variable and complex behaviour modes and failure states of strata that fall within the zone of influence of a powered support. Gale (2009) utilised the ground response curve concept in a general form to define three stages in the ground response to longwall mining, shown imprinted on Fig. 9.19. These stages are:

- Stage (i) – An elastic "intact" mass whereby the amount of support to resist ground movement is well outside the capability of any face support.
- Stage (ii) – A fractured interlocked mass that has some remaining confined strength but is

typically still outside the capacity of face supports.
- Stage (iii) – Fractured ground which starts to lose its interlocking nature, resulting in a reduction in confinement leading to unravelling and falls of ground.

The role of powered supports in this scenario is envisaged as maintaining the remaining ground strength in Stage (ii) and stopping the transition to Stage (iii). Typically, setting load is intended to provide adequate control against progression to Stage (iii).

As the fractured rock mass unravels, the situation progressively changes to a load controlled system. The stiffness of the powered supports then becomes variable, depending on where the centre of gravity of the load acts on the support. At that point, the application of a strata-support interaction curve of the type shown in Fig. 9.19 becomes problematic.

Irrespective of its static characteristics and capabilities, a longwall powered support system has limited practical value if the geometric proportions of the system and its kinematic characteristics limit contact of the canopy with the roof. It is important that the support is in good contact with the roof and that the span between the coal face and the area of application of the main thrust of the support system is small. The introduction of shearer technology that senses and remembers mining profiles along the face has aided in reducing the frequency of large vertical steps in the floor and roof caused by loss of horizon control. However, poor roof contact conditions can still occur in the presence of geological disturbances, excessive loose material on the canopy of a powered support, and roof cavities.

Rigid canopies limit the options for maximising canopy contact area when the roof profile is irregular and for applying support where it may be most needed. The two leg configuration of a shield support in association with the compensating, or canopy tilt, cylinder connecting the canopy to the flushing shield provides some potential to optimise canopy orientation and, therefore, contact area. However, this canopy orientation may not be maintained under load. It must be remembered that the effective tip distance is that distance from the face back to where the immediate roof strata comes into contact with the canopy of the powered support. Some support designs promote tip contact by curving the canopy tip upwards.

## 9.4 Operational Variables

In addition to equipment selection and mine design, there are a range of operational variables that are important for managing ground control about a longwall face. The timely and effective use of these is vulnerable to the vagaries of human performance. Therefore, they need to be underpinned by a robust Trigger Action Response Plan (TARP).

### 9.4.1 Cutting Technique and Support Configuration

There are three basic techniques for cutting coal from a longwall face, namely, **bidirectional** (**bi-di**), **unidirectional** (**uni-di**) and **half web**, and a range of permutations within each.

In bi-di mode, the face is cut from both directions to its full height and width (one web) with each pass of the shearer. This enables the AFC to be advanced immediately behind the shearer. The AFC has a limited degree of articulation and so is advanced incrementally over a distance of 15–20 powered supports, with this transition section being referred as the **snake**. It also enables the face to be double chocked (closed up) in poor ground conditions immediately after each pass of the shearer. The potential disadvantages of this cutting technique are loss of floor horizon control because it is not easy to see and monitor this horizon when cutting; poor floor cleanup leading to debris ingress under the powered supports; and extended time for the shearer to double shuffle at each end of the longwall face in order to cut out the bottom section of the face right up to the gate end.

Uni-di cutting entails mining the top section of the face from one direction and the bottom section from the other direction. This removes the need for the shearer to double shuffle at the gate ends, thereby providing for faster turn-around times, and minimises the need for operators to work in dust on the return airway side of the shearer. Ground control benefits are associated with improved horizon control and a cleaner floor. More uniform coal loading and increased cutting speed can result in cycle times that approach or exceed that of bi-di cycle cutting on longwall faces shorter than around 250 m.

Historically, the main ground control disadvantage with uni-di cutting was related to not being able to advance the powered supports until after the shearer had taken the bottom pass. The advent of powered supports that can be operated in one web back mode while still generating a high tip load and be advanced immediately after the passage of the shearer has removed this disadvantage, other than when ground conditions are so poor that the face needs to be closed up and double chocked.

Half web cutting modes involve variations on undercutting the face in uni-di mode at mid height over the middle sector of the longwall face and cutting the gate end sectors in bi-di mode using half web advances. Improvements in cycle times can translate to improved ground control. However, in weak coal, the undercut is prone to fall and to increase the tip-to-face distance.

### 9.4.2 Powered Support System Maintenance

Maintenance of the powered support system is critical to ground control on a longwall face. Matters of particular importance are:

- Condition of the hydraulic legs. The total support resistance of powered supports on a longwall face reduces in direct proportion to the number of non-functional hydraulic legs on the face. It is not uncommon for major longwall face falls to have been associated with leg fault rates exceeding 20 % (e.g. Galvin 1997b). Trueman et al. (2008) report that up to 10 % of shield legs had faults on a typical Australian longwall face. This is sufficient to adversely affect strata stability along the full length of the face. Excessive convergence, guttering and cavities can also develop on a localised scale due to load transfer from an under-performing support to its adjacent supports. The move from 4 leg chock shields to 2 leg shields has had the benefit of minimising the number of legs that have to be maintained on a longwall face. However, support performance is now more sensitive to an underperforming leg.
- Valve maintenance. Valves control a number of functions crucial to ground control on a longwall face including set pressure, yield pressure, activation of leg stages, activation of adjacent supports, positive set and positive set reactivation. Over time, they can become clogged and scoured, resulting in them operating at lower pressures than design. On a number of occasions, the poor state of valving has only become apparent after a rapid loading event when upwards of 100 or more yield valves designed to control such events have failed.
- Pressures and volumes. In theory, the hydraulic reticulation system should be capable of supplying sufficient volumes of fluid at sufficient pressure to all areas of the face. In practice, however, fluctuations in line pressure occur at times of peak demand. The midpoint of the face, where it is most critical that powered supports operate at design pressure, is the most vulnerable to insufficient supply pressure in some installations. In others, it is the tailgate third of the longwall face. Pumping rates need to be sufficient to keep up with setting times and powered support advance rates (determined by shearer cutting speed). Positive set reactivation is important to correct situations where legs may not have reached set pressure due to peak demands on the supply system. This should not be

tolerated on more than a sporadic basis as uneven set pressure distributions can induce roof instability.
- Lemniscate linkages. Lateral thrust on powered supports can cause the lemniscate linkages to become sloppy through racking and wear, allowing significant horizontal movement prior to the canopy bedding into the roof. This movement reduces and or removes lateral confinement of the immediate roof strata, increasing the potential for this strata to unravel in the tip-to-face region.
- Structural integrity. Powered support components can be subjected to eccentric loadings, concentrated loadings, point loadings, impulse loadings, cyclic loadings, and corrosive environments, all of which are conducive to deformation, wear, and fatigue failure at critical load bearing points in the structure. Often, these points may not be visible or accessible until the longwall face is salvaged. In any case, when structural failures are detected during operation, they cannot usually be remedied on the longwall face. Hence, design, fabrication techniques, inspections and maintenance of powered supports are also fundamentally important to effective strata control on a longwall face.

For reasons of both safety and productivity, it is advisable that an engineering maintenance scheme which addresses these types of issues is an integral element of the overall mine management scheme.

### 9.4.3 Face Operating Practices

Ground control on a longwall face can also be influenced significantly by operating practices and operating discipline. The following are particularly important and warrant carefull consideration when preparing a Face Management Trigger Action Response Plan:

- Rate of retreat. It is long established from total extraction mining operations that the strength of highly loaded rock, particularly sedimentary rock, can decrease over time and, therefore, the speed of extraction is a critical parameter, especially during periodic weighting events and when negotiating structurally disturbed ground. Table 9.2 summarises convergence limits and rates suggested by Medhurst (2005) as being typical for most Australian longwall mining operations.

  Based on these figures and a consideration of the extent of fracturing ahead of a longwall face, Medhurst (2005) concluded that a minimum retreat rate of 5 m/day should be maintained when mining at a height of 2–3 m, increasing to 10 m/day when operating in thicker weak coal seams.

**Table 9.2** Summary of powered support convergence limits and rates proposed by Medhurst (2005)

| Event | Convergence |
|---|---|
| Initiation of face spall | 15–20 mm |
| Cavity development | >30–50 mm |
| Overlying strata broken | >100 mm |
| Heavily weighted environments | 10 mm/h |
| Periodic weighting cycle | >20 mm/h |

- Face alignment. Maintaining a straight face alignment has long been considered important for preventing the formation of local stress raisers on the longwall face. It has a secondary strata control benefit in that it reduces the likelihood of a face stoppage due to damage to the AFC. However, some operators of faces over 250 m long report a benefit in advancing the middle third of a longwall face in periodic weighting situations. This may be related to the trajectory of mining-induced fracturing along the face.
- Horizon control. Steps in the roof and floor associated with poor horizon control can present obstructions to advancing the AFC and powered supports and prevent the support canopies from making full contact with the roof. Loss of contact with the roof effectively equates to an increase in the tip-to-face distance. Roof steps can give rise to point loads that exceed the contact strength of the roof. In

hard floor environments, floor steps can generate point loads and flexing that are of sufficient magnitude to result in structural damage to the support bases. Automatic horizon control on the shearer is an aid in managing this risk but does not eliminate it.

- Powered support advance. Support advance must not be permitted to lag behind the shearer. Automatic initiation of support advance by the shearer is a valuable control, provided that sufficient hydraulic volume and pressure are available. Programmable control circuits constitute another control, enabling powered supports to be advanced individually or in 'banks' or 'blocks' that typically comprise between two and five supports. Advances in coal cutting and clearance technologies have resulted in a significant increase in shearer speed, to the point where it is difficult to keep up with the shearer when advancing the powered supports individually. Block advance, or bank push, assists in addressing this problem but it has the disadvantage of not enabling the roof to be supported immediately upon exposure. Hence, in poor ground conditions it is advisable to slow the shearer down if necessary to enable the powered supports to be advanced on an individual basis immediately behind the shearer.

  If the immediate roof is already in a fractured state or contains the lip of a cavity that needs to be 'caught', there can be benefits in maintaining some load on the roof as the powered support is advanced. This operating procedure is referred to as **contact advance**. It can increase the time taken to advance each support and, therefore, may also require a reduction in the speed of the shearer to enable freshly exposed roof to be supported immediately.
- Setting and maintaining leg pressure. Powered supports need to be reset to the correct setting pressure after being advanced and not be permitted to drop below this pressure during a cutting cycle. Positive set and positive set reactivation are of assistance in this regard, aided by having a separate hydraulic circuit for set reactivation. Some operations employ a second 'high-set' hydraulic circuit in any case in order to increase initial set pressure to an intermediate value between nominal set and yield.
- Debris. Compaction of loose material over the top of or beneath a powered support results in additional convergence and, therefore, a reduction in support stiffness. Debris on top of the canopy can also generate point loads and reduce the area of roof that is actively supported. Debris on the floor may cause the powered supports and AFC to ride up on the loose floor material, leading to a loss of horizon control. Positive set reactivation is a control for managing these types of situations. However, a more effective control is to eliminate the debris by means such as contact advance, cutting to a different horizon, improving the dozing capability of the AFC, and clearing loose material from the floor.
- Negotiating weak roof and cavities. Risk management procedures, preferably encapsulated in Trigger Action Response Plans, should contain provisions for reverting to double chocking, conventional mode, bi-di shearing, and/or reducing mining height in a timely manner when ground conditions deteriorate. When negotiating cavities, it may be necessary to turn off the positive set system in order to maintain the attitude of the canopy. Operators need to be aware that this can result in poor set pressures across the face and no compensation for pressure loss due to leaks in the hydraulic circuitry.

Operating discipline is particularly important when it comes to stopping the face in order to install secondary support such as rock bolts, long tendons, spiles, strata binders and void fillers. Experience attests to the risk associated with continuing to mine in an attempt to 'catch the lip', rather than stopping and taking remedial action (Galvin 1996; Payne 2008). Face falls associated with attempting to outrun a situation are often vertically and laterally extensive in nature, which not only makes their recovery more time consuming but may also expose those working on the recovery operation to greater risk of injury.

- Real time monitoring of longwall leg pressure trends and yielding behaviour offers significant potential benefits in this regard because it assists in quantifying the state of face stability and provides an immediate and objective basis for risk management decision making. Hoyer (2011) and Wiklund et al. (2011) describe applications of a software package utilised for providing early warning of the development of periodic weighting and roof cavities on the basis of a leg pressure algorithm. Such algorithms can be based around average support pressures, support loading rates (pressure increase/unit time), and yield frequency per cutting cycle.
- Extended downtime. When a longwall face is to be idle for an extended period, typically more than a shift, standard work procedures should be available that detail the requirements for setting flippers and closing up the powered supports. These should be encapsulated in a Face Management Trigger Action Response Plan, which also constitutes a control for these situations.

## 9.5 Longwall Face Strata Control

### 9.5.1 Introduction

In addition to face operating practices (Sect. 9.4.3), strata control on a longwall face is a function of a range of other interactive factors that include:

- lithology and sedimentology;
- pore pressure;
- mining height;
- panel span;
- panel depth;
- interpanel pillar width;
- the static and kinematic characteristics of the powered supports;
- engineering maintenance standards; and
- the presence and nature of workings in adjacent seams.

Many of the seminal concepts of strata behaviour around a longwall face, such as those developed by Potts (1957), Salamon et al. (1972), Wagner and Steijn (1979) and Galvin et al. (1982) were based on surface to seam extensometers; surface subsidence measurements; monitoring of leg pressures and convergence on longwall faces; and observations of goaf behaviour. Subsequently, these concepts have been developed and enhanced by Kelly and Gale (1999), Gale (2004), Gale (2009) and others utilising advances in microseismic monitoring, computational techniques and stress measurement to give more detailed insight into the location and nature of rock fracturing about a longwall face.

### 9.5.2 Coal Face

The stability of the coal face is particularly sensitive to the direction, dip and density of cleats, joints and mining-induced fractures; mining height; abutment stress magnitude; and rate of mining. Cleats and joints provide pre-existing failure surfaces for face spall; delineate coal slabs and columns that are conducive to bending and buckling failure under load; and create the potential for slabs to topple onto face equipment and into the work area. While orientating the longwall face line parallel to the natural cleat and jointing direction is sometimes suggested and utilised as a control for inducing massive roof strata to cave, experience confirms that this can result in an unsafe local mining environment. It increases the risk of rib spall on the longwall face and in gateroad cut-throughs and, if a conjugate cleat or joint set is present, in the gateroad headings. It also increases the potential for face breaks and for large blocks to fall out of the roof in front of the longwall supports. Hence, for safety and operational reasons, it is generally preferable to orientate drivages at an angle of at least 20° to natural cleat and joint systems, albeit that this may aggravate spalling of pillar corners.

As mining height increases, the stiffness of both the coal face and the powered supports is reduced, resulting in increased roof convergence. The stability of the powered supports may also be reduced. There is an increased potential for face spall due to a reduction in coal strength, and for this spall to extend to a greater depth into the coal face, resulting in a substantial increase in tip-to-face distance and, hence, unsupported roof span. The dip of geological features takes on added significance because it controls the sizes of blocks that may spall from the face. Operators and equipment are exposed to higher levels of gravitational energy from face spall as mining height and block size increase. All of these impacts are magnified with increase in abutment stress. Such increases may be associated with increased depth of mining, cyclic caving or interaction with workings in the same or adjacent coal seams.

Because the depth of spall is almost invariably less at the bottom of the coal face than the top, the AFC and powered supports can be prevented from being advanced to support the increased area of exposed roof until the toe of the face has been mined. The associated time delay and potential for this operation to initiate additional face spall can aggravate the situation. Control options for safely and effectively managing these circumstances include:

- maintaining a straight face line so as to avoid localised stress concentrations (noting that some operators have reported benefits with curved faces in periodic weighting situations);
- mining to the correct horizon;
- incorporating ‘double knuckle’ flippers into powered supports to function as face sprags with an extended reach;
- closing up the face (double chocking) immediately after passage of the shearer;
- limiting abutment stress magnitude by the judicious selection of panel orientation and geometry, particularly panel width, W;
- stopping to support and consolidate the face and immediate roof before the tip-to-face distance becomes excessive (which requires operating discipline, fit-for-purpose equipment that is on-hand, and robust safe working procedures);
- incorporating facilities in thick seam powered supports for accessing the roof line to undertake consolidation and secondary support; and
- in all cases, safe work procedures that prevent operator exposure to rib spall and roof falls on a longwall face.

### 9.5.3 Floor

Abutment stress induces fracturing of the floor ahead of the face, with fractures traversing bedding and dipping back under the goaf and also running along bedding planes. Abutment stress impacts are more likely and greater when the floor strata is soft or weak or contains bands that are prone to extrude under load or to swell or disintegrate in the presence of moisture, leading to bearing capacity failure. High toe loadings on powered supports can also induce bearing capacity failure of the floor. Associated floor heave can obstruct the advance of the supports and, in the extreme case, result in supports rotating to an extent that they become unstable and topple towards the face. Floor heave can also have a serious impact on the operation of the AFC and shearer, causing the AFC to rise, relay bars to bend, and the shearer to topple towards the longwall supports. If the shearer is still able to traverse the face, horizon control may be lost. Fracturing of the floor strata can also significantly increase the potential for release of gas into the workplace from deeper seams.

Control options for safely and effectively managing these situations include:

- Optimising the design of powered supports to avoid high toe pressures. Options include solid bases and varying the canopy ratio. Floor pressure profiles may be the defining factor in determining powered support capacity.
- Incorporating base lifting rams in the powered supports.
- Leaving bottom coal to protect the floor.
- Limiting abutment stress magnitudes, by the judicious selection of panel orientation and geometry, particularly panel width, W.
- Maintaining a relatively rapid rate of face retreat.
- Limiting ingress of water into the face area by mining up dip and utilising efficient water management systems.
- Limiting influx of gas by pre-draining the mining seam and adjacent seams.
- Diluting gas make by utilising an effective ventilation system.

### 9.5.4 Immediate and Upper Roof Strata

There are no unique definitions of what constitutes immediate and upper roof strata, which may be comprised of numerous combinations of strata type, thickness and properties. However, when discussing strata response in longwall mining, it is convenient to consider the immediate roof strata as comprising the strata that constitutes the caving zone and to classify the strength of the immediate and upper roof strata as either ‘weak to moderate’ or ‘moderate to strong’.

Causes of roof cavities on a longwall face include:

- geological features;
- excessive tip-to-face distance;
- inadequate setting pressure;
- poor setting geometry, especially supports set with their tip down;
- cleat parallel to face;
- loss of horizon control;
- periodic weighting; and
- loose material above chock canopies.

#### 9.5.4.1 Weak to Moderate Strength Roof Strata

Field observations and studies supported by numerical simulations confirm that mining-induced stress can cause fracturing of weak and/or laminated strata well ahead of the longwall face, with fracture network intensity increasing towards the face and resulting in the strata caving readily immediately behind the longwall supports. This is confirmed by the outcomes of seismic monitoring, shown in Fig. 3.16d, that was undertaken by Hatherly et al. (1995) and Kelly and Gale (1999) in a weak roof and floor environment about a longwall panel. Gale (2004) utilised a two-dimensional FLAC model to simulate the behaviour of strata and fluid pressure along a longitudinal plane running down the centre of this longwall panel. The derived fracture network is shown in Fig. 9.20. It was concluded from these combined studies that:

- fracturing of the roof and floor strata occurs well ahead of the mining face and is not related to the caving process behind the supports;
- the dominant initiating failure modes in weak strata ahead of the face are shear fracture of rock mass and shear along bedding (Fig. 9.20);
- the extent of bedding plane shear ahead of the longwall face is variable but typically extends over large distances, often in excess of 100 m;
- in general, fracture size is variable, however, shear fractures tend to be limited to less than a couple of metres and form in an incremental manner rather than in one large event;
- tensile initiated fractures may develop ahead of the face in response to bending moments but these fractures are generally confined to stronger upper strata;

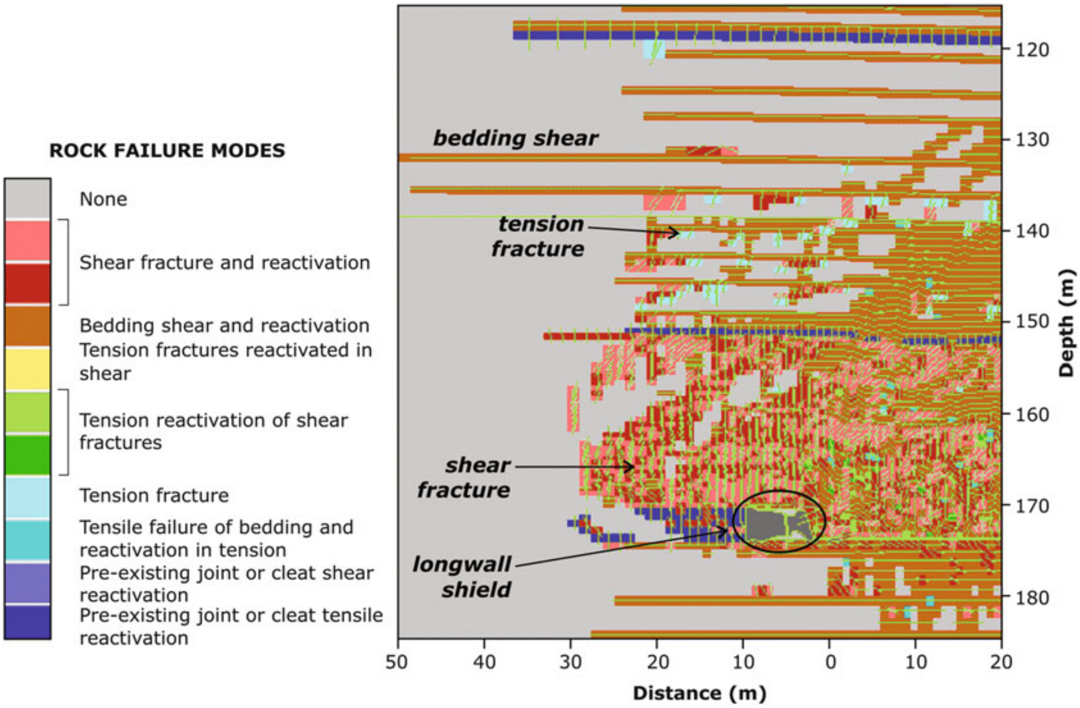


**Fig. 9.20** Rock failure modes and fracture orientations in weak to moderate strength roof as predicted by two-dimensional numerical modelling (After Gale 2004)

- few new fractures are generated during the caving process; and
- the high frequency of fracturing prevents the accumulation of large amounts of stored strain energy, as reflected by the low seismic magnitude of fracture events (typically less than −1 on the Richter scale).

The various monitoring and analyses highlight that good forward roof support is critical in weak strata conditions. In subsequent analysis of the performance of a 2 leg shield in weak to moderate roof sections, Gale (2009) concluded that:

- The yield capacity to control the caving line and provide confinement to the fractured material is recommended to be in the 1–1.1 MN/m$^2$ (100–110 t/m$^2$) range, with a set of approximately 0.8 MN/m$^2$ (80 t/m$^2$).
- The canopy balance is recommended to be less than 2.4 and, preferably, less than 2. In most instances this relates to a tip to leg distance of 3–3.5 m, corresponding to the reaction point of the legs being less than 0.7, and preferably less than 0.6, of the canopy length back from the face.
- The tip-to-face distance to maximise roof integrity and limit dilation should be less than approximately 0.6 m. The smaller the distance, the better the result.
- A rigid canopy offers benefits over a hinged cantilevered canopy.

Increased convergence arising from the powered supports going into yield is conducive to roof scaling, slabbing roof, and guttering, resulting in cavities over the top of the powered supports as they are advanced. These cavities can limit the extent to which the canopies come into contact with the newly exposed roof and prevent the supports being set at their specified pressure, thus encouraging the formation of further cavities. Slow rates of retreat also compound this situation.

Control options for safely and effectively managing weak and friable immediate roof situations mirror many of those for managing coal face stability and include:

- restricting tip-to-face distance to a minimum;
- powered supports with a high tip capacity;
- contact advance of powered supports;
- flippers with a tilting capacity to provide immediate forward support to the roof and/or face;
- leaving top coal to prevent slabbing of weak, friable immediate roof strata;
- closing up the face (double chocking);
- limiting abutment stress magnitude by the judicious selection of panel orientation and geometry, particularly panel width, W;
- stopping to support and consolidate the face and immediate roof before the tip-to-face distance becomes excessive; and
- incorporating facilities in thick seam powered supports for accessing the roof line to undertake ground consolidation and secondary support.

#### 9.5.4.2 Moderate to Strong Strata

The presence of stronger strata units in the immediate roof might reasonably be expected to result in improved longwall face conditions. However, should these units or strata higher up in the roof sequence be sufficiently massive to result in cyclic caving, then periodic weighting becomes a concern. These cycles typically occur at intervals of 10–30 m, but may exceed 70 m in some circumstances. Periodic weighting may also develop in overburden sections that have a relatively uniform shear strength sufficient to allow a limited span between the longwall face and the goaf to develop (Gale 2001).

The impact of a massive stratum on the severity and frequency of cyclic loading, face conditions and surface subsidence is a function of the thickness and material properties of the massive stratum; its distance above the mining horizon; its depth below surface; the width of the extraction panel; and face control measures. Generally, the closer a massive unit is to the extraction horizon, the less thick it needs to be to result in periodic weighting. Periodic weighting can be influenced by the behaviour of competent beds up to 70 m or more above the seam (reference, for example, Wagner and Steijn 1979; Mills and O'Grady 1998; Trueman et al. 2008; Wiklund et al. 2011).

Periodic weighting gives rise to zones of intense fracturing in the coal face and immediate roof and floor strata and slabbing of the coal face. Significant convergence of the powered supports is associated with caving of the cantilevered strata. Slabbing of the coal face both removes support to the immediate roof and increases its unsupported span, thereby increasing the risk of local roof falls.

The risk of roof falls is elevated further because periodic weighting is also usually associated with discontinuous subsidence, whereby a gap develops at the base of the bridging strata. This results in the goaf strata being compressed only by the weight of the parting and not by the total weight of the overburden, resulting in a significant reduction in the lateral constraint provided to fractured strata in the vicinity of the face. The combination of high face stress, extensive fracturing of the coal seam and roof strata, and the low lateral stress in the goaf, leads to a potentially dangerous situation whereby massive blocks formed by mining-induced fractures can slide out of the roof on the longwall face (Wagner 1994). The problem becomes more severe with increase in mining height. In some instances, a detached block can fall onto the back of the powered supports during a weighting event. The resultant force of the slab hitting the goaf shield has the capacity to push the support forwards into the AFC and face (Hookham 2004).

Creech (1996) observed that following a periodic weighting event, mining-induced shear planes dipping back over the powered supports were present for the next four to six metres of extraction. Operational experience confirms that the risk of a fall of ground is not immediately reduced once the strata caves. On the contrary, the risk is often elevated until the face has been advanced several metres through the mining-induced fracture zone because the caving event results in removal of confinement to the shattered and failed roof and coal face material in this zone, allowing it to unravel.

A number of models have been proposed to explain periodic weighting. Following on from Beer and Meek (1982), Wold and Pala (1986) applied voussoir beam theory to analysing periodic weighting associated with a massive sandstone overlying a relatively friable 10–15 m thick lower roof at Ellalong Colliery. Frith and Creech (1997) initially proposed a form of detached block model but later subscribed to a voussoir beam model proposed by Seedsman and Stewart (1996). Gale (2004, 2009) utilised the FLAC computational code to model behaviour down the centreline of extraction panels. All are two-dimensional approaches and have their limitations but, nevertheless, are useful for conceptualising behaviour and undertaking parametric analysis of periodic weighting, which is fundamentally a three-dimensional behaviour.

Gale (2004) simulated the nature of fracturing and caving associated with a massive sandstone unit having a UCS of approximately 40 MPa that immediately overlaid a coal seam (Fig. 9.21a). This was complemented with measurements of cyclic loading of supports (Fig. 9.21b) and monitoring of overburden caving using surface extensometers when the seam was mined. It was concluded that:

- a block of massive strata begins to form in the immediate roof early in the caving process;
- the massive block develops bending stresses in response to overburden subsidence onto the goaf;
- failure is initiated in the upper section of the cantilevering block and then progresses rapidly down towards the seam;
- resistance to convergence is lost;
- face convergence may be ‘instantaneous’ or occur over a number of shears, depending on lithology; and
- overburden ‘rebound’ rather than gravity dropout of an isolated block may be the

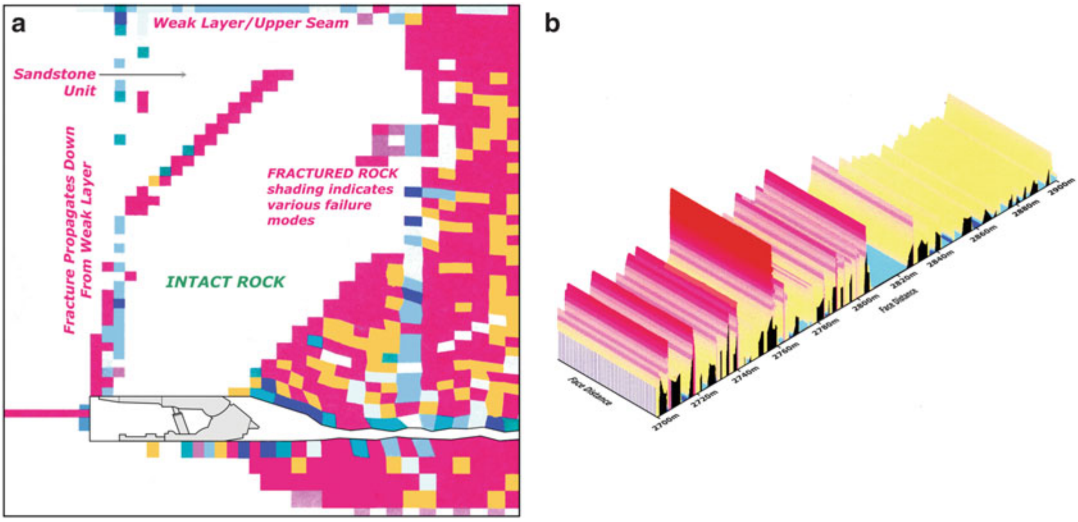


**Fig. 9.21** An example of two-dimensional numerical modelling applied to the simulation of fracture mode and orientation associated with longwall mining beneath a massive sandstone unit, complemented with monitoring of longwall powered support pressures when mining beneath the unit (After Gale 2004). (**a**) The nature of fracturing and caving as determined by numerical modelling. (**b**) Monitored rates of increase in powered support pressures for the situation modelled numerically in (**a**), with the red bands corresponding with episodes of cyclic weighting

principal driver of convergence, thereby resulting in a finite displacement, typically in the range of 0.1–0.6 m.

The significant influence of the composition of the immediate and upper roof strata on strata behaviour around the longwall face is illustrated by comparing Fig. 9.20 with Fig. 9.21. Field observations and microseismic monitoring (Fig. 3.19b) revealed a strong trend for periodic weighting events to concentrate around cut-throughs. Frith (2005) reported that at South Bulga Colliery, Australia, no major weightings were ever experienced outside of a few metres from a chain pillar cut-through.

The impacts of periodic weighting on face stability and equipment can range from nuisance value to complete loss of the face. During the mining of Longwall 5 beneath a massive immediate roof at Newstan Colliery, Australia, periodic weighting resulted in 14 falls of ground at 35–40 m intervals that extended up to 70 m along the face, up to 6 m ahead of the face, and over 10 m into the roof (Hebblewhite and Simpson 1997). Blocks in excess of 6 m long and 1 m wide fell onto the AFC or bridged between the face and the powered supports, which on occasions converged 1–1.5 m over a period of one to two shifts. Phalen Colliery, Canada, experienced over 1 m of convergence in less than four hours on a longwall face (MacDonald 1997), while a convergence rate of 15 mm/s was recorded during a dynamic event at Churcha West Colliery, India, that resulted in some 1.5 m of closure in less than one hour and the destruction of 23 chock shield supports (Gupta and Ghose 1992).

In an attempt to control periodic weighting, some operators have replaced powered supports with higher capacity units. While this offers benefits, the improvements are generally limited and, because of lever arm effects, are not in proportion to the increase in nominal support capacity. For example, analysis by Galvin (1997b) showed that when the capacity of 4 leg chock shields working under a 6.3 m thick massive immediate roof was increased from 8 MN to 9 MN (800 t to 900 t), the extra overhang that could be supported before the powered supports went into yield was only 1 m.

Effective mine design controls for mitigating periodic weighting are either to increase panel width such that the massive strata caves soon after the commencement of panel extraction and then at very short and regular intervals thereafter, or else to limit panel span such that the massive strata bridges the panel indefinitely. The manipulation of panel width to control periodic weighting has been applied very successfully for decades in South Africa on the basis of the Galvin dolerite sill failure span formula (Galvin 1983). However, if panel span is deliberately restricted, care is required to ensure that this does not result in excessive abutment stress throughout the life of the panel or in windblasts. At Coalbrook Colliery, South Africa, longwall panel span was designed to be either greater than 200 m in order to induce failure of the overlying dolerite sill, or else less than 120 m in order to control abutment stress (Henderson 1980). While changing longwall span at Newstan Colliery from 226.5 m to, initially, 90 m and, subsequently, 150 m proved very successful in mitigating severe face instability associated with periodic weighting, it resulted in shallow caving of the nether roof that generated violent windblasts (Hebblewhite and Simpson 1997).

The trend in longwall mining is towards wider faces, or panel spans, made possible by advances in coal clearance technology, in particular synchronised multiple drive motors for AFCs. Wider faces are attractive because they reduce gateroad drivage metres and down time associated with longwall moves and the extent of the surface affected by differential subsidence. However, they increase financial risk associated with an underperforming installation. From a ground engineering perspective, it might be concluded that once panel width-to-depth ratio becomes supercritical, an increase in face width will have little impact on ground behaviour other than that the length of longwall face subjected to maximum abutment stress may increase and the longer time between shears increases the opportunity for the face to deteriorate. However, early experience with a 400 m wide longwall face in Australia indicates that abutment stress impacts extend further outbye of the face than those associated with narrower supercritical width faces.

Otherwise, once the span of a longwall reaches the critical width, little detailed consideration is usually given to the impacts of further increases in longwall panel width. However, careful consideration has to be given to a reduction in longwall panel width below its critical span in order to avoid the operation being subjected to high abutment stress throughout the life of the panel. Furthermore, a relatively small change in panel width-to-depth ratio or geology can result in a step change in surface subsidence, as evidenced in Fig. 3.14.

The reader is referred to the range of approaches to determining panel span presented in Sect. 3.3.3. Once again, the situation is similar to that in pillar extraction in that semi-empirical and analytical models can provide reasonably accurate estimates of the span required to induce full caving and subsidence if calibrated to site-specific data. Appropriately chosen and constructed numerical models can be valuable for quantifying abutment stress magnitudes and distributions as a basis for selecting mining span, but outputs can also be unreliable. Therefore, numerical modelling outcomes should be used as an aid and supported by parametric and sensitivity analysis, rather than being accepted as absolute and correct. Irrespective of the desktop approach taken to design, historical field performance and local operational experience are invaluable for determining mining span, especially in situations where there is potential for cyclic loading and/or windblast.

Precursors to cyclic loading events can include:

- audible noise or ‘bumping’ of surrounding strata, sometimes correlating with significant coal face spalling or ejection of coal from the face and with proximity to a cut-through;
- guttering at the face/roof intersection;
- face spall;
- roof spall;
- water make from the roof;
- an increased rate of rise from set to yield pressure in powered supports;
- an increased rate of convergence of powered supports; and
- a dynamic loading event.

Controls to minimise the occurrence and impacts of periodic weighting are:

- Mine design
  - The minimum dimension (width) of an extraction panel should be sufficiently large to induce caving of massive strata very soon after commencement of extraction and at short intervals thereafter, or else sufficiently narrow to prevent the onset of caving and to limit abutment stress.
- Powered support design
  - The supports should make provision for minimising the unsupported span from the point of effective support load application to the coal face. This may require some form of articulation at the front of the support.
  - Supports should have a high tip support capacity.
  - Support resistance should be maximised by minimising the total supported roof area.
  - Supports should incorporate face sprags where working height permits.
  - Yield valves should be of a rapid release type.
- Powered support operation and maintenance
  - Support hydraulics should be maintained to a high standard with minimal hydraulic leaks.
  - Hydraulic pumps should not be shut down during a periodic weighting event.
  - Adequate volumes of hydraulic fluid at the correct pressure need to be available.
  - Supports should incorporate guaranteed set.
  - Supports need to be set at optimum pressure being, typically, at least 80 % of yield.
  - Positive set should only be turned off in areas affected by cavities.

  - Yield valves should be maintained in good condition and operate at design relief pressure.
- Face operation
  - Since the strength of rock can reduce over time, it is important to retreat the face regularly so that abutment stresses have less opportunity to cause fracturing of the coal face and the immediate roof and floor strata in the vicinity of the coal face.
  - The face should be maintained in a straight alignment (noting once again that some operators report a benefit in the centre of the face being in advance of the gate-ends).
  - At high mining height, flippers (sprags) should be used at all times. These will not only stabilise the face, but also prevent broken material from flushing onto the AFC.
  - Subject to not inducing excessive face spall, mining height should be maximised prior to an anticipated weighting event in order to accommodate yielding of powered supports.
  - Rate of face retreat should be maximised but only to the extent that it remains regular and controlled.
  - Maintenance should not be scheduled and the face should be worked around the clock during the event.
  - Powered supports should be advanced individually just behind the shearer so as provide support immediately to newly exposed roof. Bank push is not advisable.
  - A rapid rate of face retreat should be maintained for at least 3–6 m after relaxation of face pressure in order to prevent unravelling of shattered, unconfined face and roof strata.
  - If face spalling or fallen roof material is excessive or retreat rates are rapid, coal clearance capacity may need to be maximised by shutting down other belt systems.
  - In the event of excessive face spall or the face having to stand for any period of time, the face should be closed up.
  - Adequate supplies of suitable secondary support, strata consolidation products and void fillers should be on hand in the event that the face has to be stopped; the tip-to-face distance becomes excessive; or face or roof control is lost.
  - Secondary support measures need to be implemented as soon as face control begins to be lost. Support pressure monitoring algorithms based on factors such as loading rates, yield frequencies, time weighted average pressures and number of affected powered supports (for example, that described by Hoyer 2011), can provide early warning in this regard.
- Outbye services
  - Allocate labour to outbye services to minimise disruption to longwall face operations, especially those caused by conveyor belt stoppages and loss of electric and hydraulic power supplies.

The microseismic monitoring associated with Fig. 3.19 gives insight into a number of aspects of strata behaviour for the case of multiple 200 m wide longwall faces located at a depth of around 500 m. Aspects of particular note include:

- the majority of fracturing (low frequency events) extended to a height of 50–70 m above the seam and to a depth of 80–90 m into the floor;
- cyclic failure was not symmetric about the longwall face but was biased from mid-face to the tailgate;
- mining reactivated strata failure beneath the chain pillars of the previously extracted panel, up to 300 m away; and
- mining activated a strike-slip structure in the maingate (high frequency events) when it was still more than 300 m from the structure.

## 9.6 Installation Roadways

Geological structure, horizontal stress, seam dip, cleating and jointing are some of the in-seam factors that give rise to preferred mining directions. In longwall mining, it is usual to orientate the panels so that neither the headings nor the cut-throughs are orientated in the least preferred direction. Headings can be biased towards the favoured mining direction while still limiting exposure of the cut-throughs to the poorest ground conditions if the cut-throughs are not driven at 90° to the headings (reference Sect. 9.2.2). At the inbye end of the longwall panel, however, there is no option but to drive the longwall face installation roadway at 90° to the headings. To provide sufficient space to safely manoeuvre the longwall equipment, the width of this roadway typically ranges from 7 to 11 m, depending on the size of the longwall face equipment.

The drivage and support of a wide roadway presents an elevated risk due to the increased likelihood of ground instability and the potentially high safety and business related consequences associated with instability of such a critical roadway. Therefore, drivage methodology and support design warrant careful consideration.

In benign ground conditions, the installation roadway may be driven in a single pass. This offers considerable operational advantages associated with coal clearance, ventilation, advancing services, installing support, and maintaining a flat roof horizon. Most often, however, strata control considerations require the roadway to be driven in two passes, and sometimes three, in order to restrict unsupported span. This usually requires subsequent passes to be cut to a lower roof horizon to avoid damaging roof support already installed in the roadway.

If the installation roadway has to be driven at some acute angle to the direction of an elevated horizontal stress field, a lateral stress shadow will be induced in both flanks of the drivage. The protection that this provides to the second pass depends on:

- stress magnitude, strata strength and strata stiffness;
- which side of the first pass the second pass is driven; and
- the direction of drivage of the second pass.

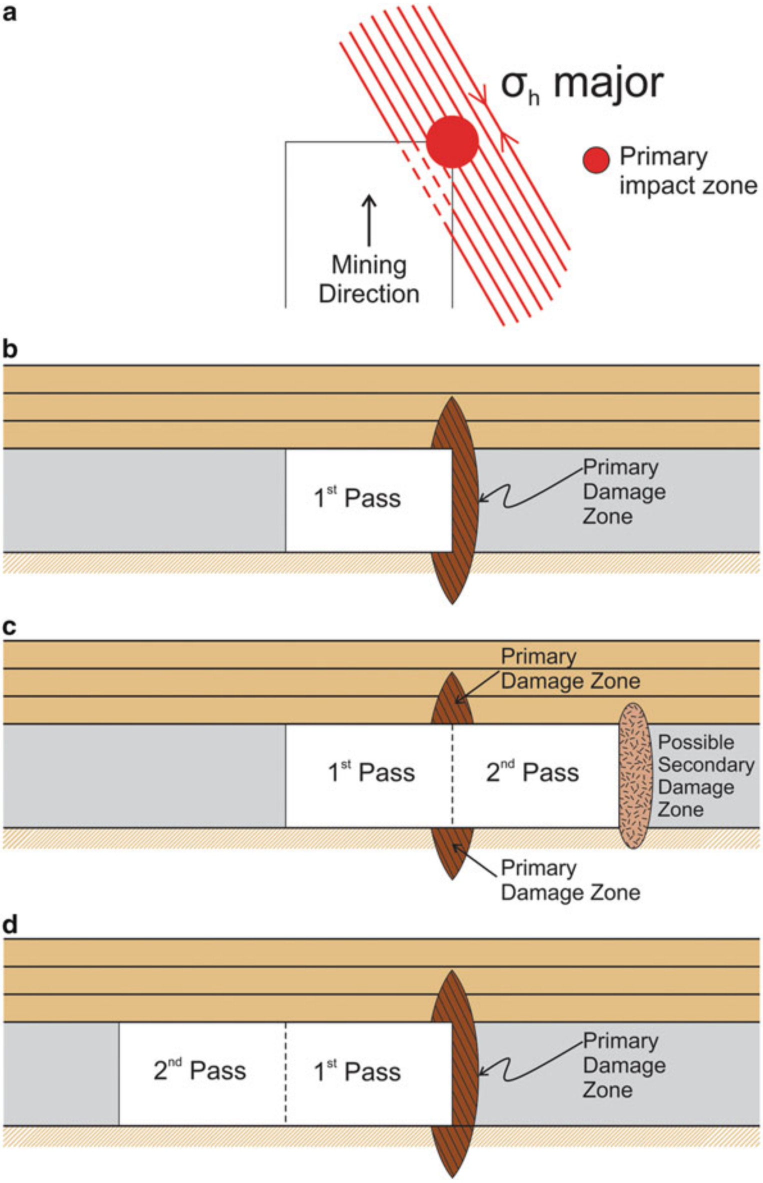
If, in the example shown in Fig. 9.22, the second pass was to be driven in the same direction as the first pass and to the right of it, the right hand corner of the new drivage would once again be the leading corner in penetrating the lateral stress field and so might be impacted, albeit perhaps not to the same extent as during drivage of the first pass. On the other hand, if the second pass is driven to the left of the first pass, the leading corner is immediately adjacent to the void of the first pass and, therefore, is well within a lateral stress shadow and protected by ground support already installed in the first pass. The left hand side of the second pass does not come to be exposed to the lateral stress since mining on the right hand side causes this stress to be redistributed in advance of mining the left hand side. If the second pass is driven from the opposite direction to that of the first pass, the situation just described is reversed.

Consideration also needs to be given to the ultimate location of the primary damage zone in the completed installation roadway, with the options being for it to be located at the face ribside; towards the centre of the roadway; or, preferably, at the goaf ribside. The location of the primary damage zone towards the centre of the installation road can introduce additional operational and safety issues for face workers when mining through and supporting the damaged zone on the second pass. In particular:

- it is problematic if the free edge of the roof of the second pass will remain intact in the structurally disturbed conditions;
- there are operational and quality assurance challenges associated with drilling long holes and achieving effective anchorage in ground that is already fractured; and
- a roof fall can undermine and render ineffective the support installed in the damaged zone during mining of the first pass.

A factor not to be overlooked is damage to the floor, which can cause serious operational problems given the large ground forces and multiple movements of equipment associated with relocating longwall equipment, especially in the presence of water. These problems are most severe if the primary damage zone is located

**Fig. 9.22** Location of potential damage zones in an installation roadway driven at an acute angle to the direction of an elevated horizontal stress field. (**a**) Plan view of 1st pass, (**b**) Cross-section through 1st pass, (**c**) 2nd pass driven adjacent to stress field, (**d**) 2nd pass driven in stress shadow


a
σh major
Primary impact zone
Mining Direction
b
1st Pass
Primary Damage Zone
c
Primary Damage Zone
1st Pass
2nd Pass
Possible Secondary Damage Zone
Primary Damage Zone
d
2nd Pass
1st Pass
Primary Damage Zone

towards the centre of the completed installation roadway.

A higher degree of uncertainty is associated with the design and performance of roadway reinforcement systems when the primary damage zone is located in the centre of an installation road rather than in a supported and partially confined state in a ribside. If the situation deteriorates to the point where standing support is required, this support needs to be installed towards the centre of the roadway, where it then presents a serious obstruction to the installation of the face equipment and an additional risk when the time comes to remove it. Hence, ideally, the installation roadway should be driven in a direction that results in the lateral stress induced damage being located at the rear of the powered supports. For the example shown in Fig. 9.22, the optimum situation would be to drive the second pass on the left hand side of the first pass, with longwall mining retreating to the left.

Caution is required if support design is based purely on empirical data sourced from other mines and when the design process has limited regard to behaviour mechanics. The amount of convergence that develops during mining of the first pass is an important consideration when determining support requirements for the second pass. However, regard must also be had to the potential for ground behaviour mechanisms to change during subsequent mining passes. Some support design procedures have a reliance on criteria that have limited, if any, regard to stress paths or to the mechanical behaviour of the ground support elements and the surrounding rock mass. Reinforcement Density Index, discussed in Sect. 7.3.5, is an example of one of these criteria.

Controls to assist in managing ground stability when driving installation roadways include:

- Optimising mining direction to minimise exposure of installation roadways to elevated horizontal stresses.
- Minimising installation roadway width. This includes avoiding the holing of an installation roadway and, therefore, the formation of an intersection at any point along its length.
- Driving the installation roadway in at least two passes so that it is already in a partially reinforced state when full span is achieved.
- Numerical modelling supported by monitoring data to aid in identifying behaviour mechanisms, stress paths and support requirements, and to support empirical design procedures.
- Monitoring of strata response during each phase of the drivage process and timely processing and evaluation of monitoring data.
- Installing powered supports as soon as possible after the completion of drivage.
- Delaying the driving of the second pass or installing a higher density of support to counteract any creep behaviour if the installation roadway is to stand for an extended period of time.
- Trigger Action Response Plans which provide for timely identification and response to deviations from anticipated behaviour during each pass.
- Contingency Plans which provide for the necessary materials, equipment and competent personnel being on hand to respond to triggers.
- Utilising powered supports as a temporary support measure in critical situations by installing them in a longitudinal line down the centre of the installation roadway.

Sometimes it is unavoidable that an installation roadway has to be orientated in an adverse direction to a high horizontal stress field. A control option in these situations is to place the roadway in the stress shadow of an adjacent drivage (Figs. 5.4, 5.5, and 5.6). This drivage may take one of three forms which, in order of increasing reliability and effectiveness, are:

- A conventional roadway supported in the standard manner for the mine. This approach has met with limited success.
- A conventional roadway with minimal support such that it remains in a safe condition during drivage but promotes roof softening and may fall after mining has ceased in the area.
- A sacrificial roadway comprising a conventional roadway supported in the standard manner and then lifted off (widened without installing additional support) on the retreat so as to encourage caving to a substantial height, typically to at least two-thirds of the overall roadway width (Fig. 9.23).

The concept of a sacrificial roadway, or stress relief roadway, has proven highly effective at some mines (reference, for example, Galvin 1996 and Doyle and Gale 2004). Figure 9.24a shows the condition of the cut-through leading to a longwall installation roadway. In this particular case, stiff stone bands interbedded with coal plies were prone to shear and dilate under the effect of high horizontal stress and rapidly drive down the

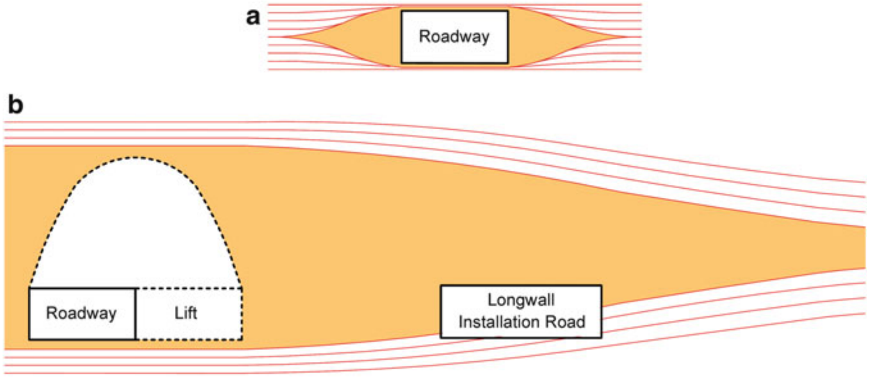


**Fig. 9.23** An illustration of the concept of driving an installation roadway in the stress shadow of a sacrificial roadway. (**a**) Horizontal stress contours around a roadway, (**b**) Location of longwall installation roadway in a zone of reduced horizontal stress due to shadowing effect of sacrificial roadway

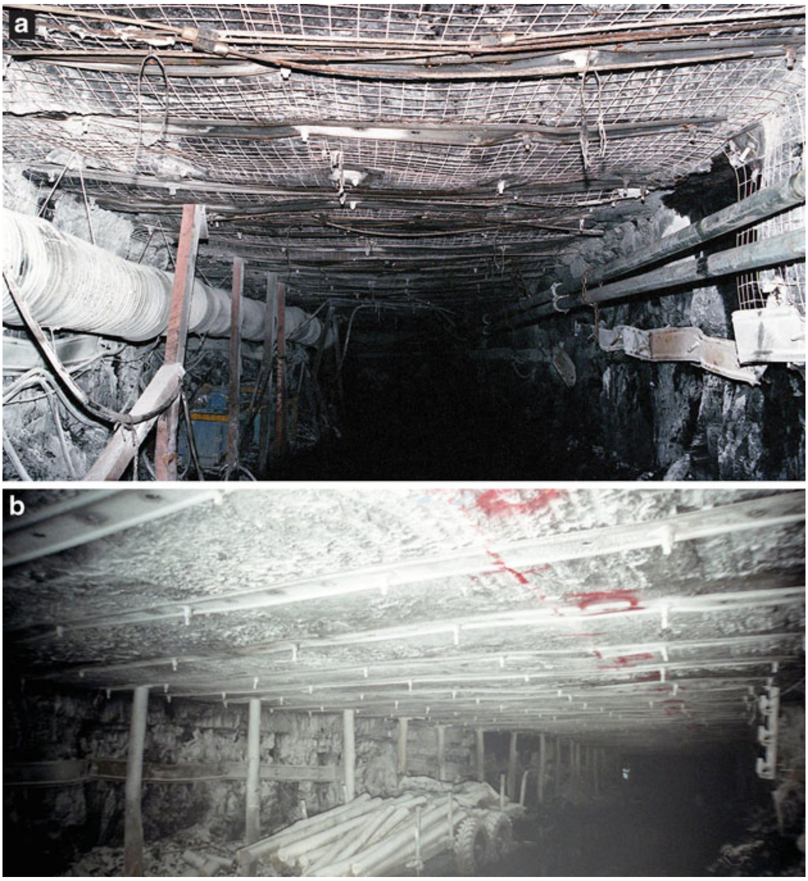


**Fig. 9.24** An example of the beneficial effect on ground control of utilising a sacrificial roadway to create a lateral stress shadow in a high horizontal stress environment. (**a**) A 4.8 m wide cut-through driven at ~65° to the regional major horizontal stress, which was $\sigma_1$ (**b**) An 8 m wide longwall installation roadway driven from the cut-through shown in (**a**) after the formation of a 10 m wide caved sacrificial roadway some 9 m further inbye

immediate roof. Drivage of the installation roadway was delayed until after a parallel roadway had been driven, widened and caved (lifted off) some 9 m inbye, so as to place the installation roadway site in a stress shadow. Figure 9.24b shows the remarkable improvement in conditions and ground support requirements when the installation roadway was driven.

While sacrificial roadways are effective in redirecting horizontal stress away from an installation roadway, this can introduce a new risk in the form of tensile failure, especially in jointed ground. Removal of the horizontal stress results in unclamping of the jointed ground, enabling it to fall without warning. Abnormal water make from the roof can be one of the few warning signs of impending tensile failure. This failure mode can present an elevated risk because:

- most often, roof reinforcement patterns in underground coal mining are not designed to control a tensile environment;
- failure is more likely to develop suddenly due to the absence of secondary reinforcement in the form of long tendons;
- the destressed installation roadway is often driven at full face width in a single pass rather than in two or more passes, therefore resulting is a lower likelihood of detecting signs of impending failure in time to respond effectively;
- standard monitoring instrumentation may not detect the onset of tensile failure at all, or in time; and
- mine workers are likely to be conditioned to recognising and responding to signs of compressive stress rather than tensile stress.

## 9.7 Pre-driven Roadways Within a Longwall Block

Pre-driven roadways can comprise:

- Stubs driven a limited distance into a longwall block.
- Existing bord and pillar workings.
- An excavation formed as an outcome of extracting a dyke ahead of a longwall face.
- One or more roadways driven across the full width of a longwall block. Sometimes these are pre-planned to facilitate ventilation and gateroad drivage from multiple locations in very long longwall panels and are aligned so as not to trend parallel to the longwall face. Often they are a legacy of past mining and trend parallel to the longwall face.
- A longwall recovery roadway driven parallel to the longwall face over its full length.
- Short longwall recovery stubs driven some distance in from each gate end.

Fundamentally, longwall mining into a pre-existing excavation is a practice that is contrary to ground control principles. It is undertaken because, if successful, it may offer high financial rewards in terms of continuity of coal production; cost savings in not having to relocate the longwall face around the excavation; and, in some situations, increased resource recovery. However, the risks can be high, especially when mining into a pre-driven longwall face recovery roadway. These risks relate not only to financial loss arising from equipment damage and extended loss of production if face stability is lost but, most importantly, to the health and safety of the mine workers, both at the time of losing ground control and during recovery operations.

International experience suggests that the failure rate associated with mining into pre-driven longwall recovery roadways is of the order of 10 %. This is high and all the more noteworthy because of the high consequences associated with failure. Past troublesome and/or unsuccessful cases such as those discussed by Gardner (1987), McKensey (1988) and Klenowski et al. (1990), and subsequent unpublicised events, highlight the need to carefully consider the risks associated with this practice.

Some unsuccessful outcomes have been associated with design methodologies that do not recognise or properly evaluate all the primary controlling variables because of their empirical nature, especially when they rely to a considerable degree on curve fitting to empirical data. In these later cases, shortcomings have been compounded on occasions by misplaced confidence in statistical analysis. Given the risk profile of pre-driven roadways (probability of a strata failure and consequences of this failure), it is strongly recommended that the design of a pre-driven longwall recovery roadway is underpinned by an applied mechanics approach, supported by appropriate numerical modelling.

### 9.7.1 Generic Types and Mining Practices

Stabilisation of pre-driven roadways in preparation for longwall mining consists of one or a combination of:

- reinforcement of the roof and ribs using short and long tendons and mesh;
- standing support, such as large diameter timber legs, timber and cementitious based chocks and packs, and monolithic cementitious cylinders; and
- backfilling, most often with a weak cohesive mixture of cement, flyash and sand or coal.

Mining into an excavation formed as a result of dyke extraction warrants additional care because the dyke contact surfaces constitute discontinuities extending well into the upper roof strata. The discontinuities disrupt the transmission of mining-induced stress, giving rise to stress concentrations, and providing potential failure planes for large wedges to drop out of the roof.

Van der Merwe (1988) reported on three instances at a mine in South Africa where a dolerite dyke was extracted ahead of 200 m wide longwall faces at a depth of around 140 m, corresponding to a panel width-to-depth ratio, W/H, of 1.4. The dyke meandered between 0° and 15° off the line of the longwall face, such that only portions of the dyke were exposed at any point in time. The immediate roof of the coal seam was weak and highly laminated. On the first occasion, the excavation was supported successfully with timber chocks (packs). The second occasion was also successful, this time utilising a reduced pack spacing. On the third occasion, the excavation was supported with timber props. The floor of the longwall was some 300 mm lower than that of the pre-driven excavation when it was intersected, necessitating secondary blasting. The slow progress resulted in steadily worsening conditions and eventually a face break occurred. Although the longwall face was recovered, very high costs and production losses were incurred.

Subsequently, the same dyke was removed ahead of a longwall face in a lower seam. The roof of the excavation was cable bolted and the ribs were supported with wooden dowels before being backfilled. Despite mining only 10–12 m directly beneath the interpanel pillars in the upper seam and several extended production delays of up to a week during extraction through this area, no instability problems were encountered.

Minney (1999) reported on the successful extraction of a dyke trending sub-parallel (~8°) to two longwall faces in a competent immediate and upper sandstone roof environment at New Denmark Colliery, South Africa. The longwall face width was reduced from 200 to 120 m to modify the behaviour of the upper roof strata which contained a massive sandstone unit some 21 m thick. The dyke excavation was supported with fully encapsulated cables and the tailgate was kept 10 m in advance of the maingate so as to hole into the excavation progressively. Success was attributed in part to the presence of massive sandstone roof.

Other experiences at this mine serve to illustrate the critical role that the stiffness of the mining system plays in determining the success of mining into pre-driven roadways. These include the longwall mining of nominally 80 m wide panels of standing bord and pillar workings reported by Galvin et al. (1991) and Bruins (1997), shown in Fig. 8.18 and discussed in Sect. 8.3.2.4, and the utilisation of only 9 m wide chain pillars between longwall panels (Galvin 1997a; Minney and Karparov 1999). Figure 8.18 reflects the benefit of maintaining a stiff loading environment by restricting the panel width-to-depth ratio.

Jones (2008) reported on the successful extraction of an igneous plug ahead of a longwall face in Australia, where longwall mining of standing pillars has also been undertaken on a small scale. At Homestead Colliery, a 3 heading development that extended about two-thirds of the way across a 200 m wide longwall block was backfilled with a cement, sand and flyash mix prior to the successful passage of the longwall face (Grice et al. 1999).

A number of mines have successfully extended the width of longwall panels by extracting adjacent standing pillars. Van der Merwe (1989) reported on a longwall operation at Bosjesspruit Colliery, South Africa, that extracted a row of pillars at the tailgate, with success being attributed to the presence of a very competent sandstone roof.

Cordeaux Colliery, Australia, successfully extended the width of a longwall panel some 45 m by also extracting a line of standing pillars at the tailgate end of the face. The roof of the 15 year old workings was re-supported with a combination of monolithic cementitious cylinders and pretensioned cables up to 6 m in length. The ribs were re-supported with 1.2 m long cuttable rib bolts and synthetic mesh and the maingate face end was maintained in advance of the tailgate end. Fisher (2001) reports that excessive noise and rib convergence occurred in the 8 m to 4 m zone from holing and, at a fender width of 2 m, the cut-throughs exhibited signs of failure. Floor heave occurred in the roadways about to be holed. Once the fender was removed, there was a marked acceleration in roof displacement of up to 10 mm/h while the powered supports were in yield. Displacement tapered off when the face had advanced about half way across the pre-driven roadways.

All these case studies involve loading environments that vary substantially from those applying to most pre-driven longwall recovery roadways. For example, panel width-to-depth ratio, W/H, was deliberately restricted in some cases. Others involve pre-driven roadways that were narrow; and/or exposed to abutment stress in a confined state; and/or located towards one end of the longwall face. It is these types of considerations that make it strongly advisable from a risk management perspective for design to be based on a mechanistic approach supported by sensible numerical modelling rather than only on a purely empirical approach.

### 9.7.2 Pre-driven Longwall Recovery Roadways

A pre-driven longwall recovery roadway can comprise a short stub driven from a gate end or a roadway that trends parallel or very near parallel to the face line across the full width of a longwall panel. Driving a longwall into a pre-driven roadway presents an elevated risk, especially in the case of a roadway that extends over the full width of the longwall panel, because:

- The pre-driven roadway causes an increase in abutment stress.
- There is potential for a higher density of mining-induced fracturing around the longwall recovery roadway because the strata are effectively unconfined when subjected to the approaching abutment stress front of the longwall.
- The width-to-height ratio of the pillar, or fender, between the longwall face and the pre-driven roadway is progressively reduced along the full width of the longwall face, such that:
  - the stiffness of the fender is also progressively reduced, resulting in increased seam convergence and, therefore, increased mining-induced fracturing and load transfer onto the powered supports and the outbye coal face;
  - face spall leading up to fender failure increases the tip-to-face distance at roof level but may leave a wedge of material at floor level that prevents the powered supports immediately being advanced to control this situation;
  - when the fender ultimately fails, there is a step increase in effective tip-to-face distance, increasing the potential for unravelling of the immediate roof and face falls;

  - conditions may be conducive to the fender failing in a sudden and violent manner; and
  - loss of strata control may extend along the full length of the face.
- Prior to the fender failing or being extracted, fender stress may initiate failure of the fender foundations, resulting in the fenders punching into the roof or floor and inducing roof falls and floor heave that present an impediment to advancing the powered supports.
- In the event of a face break, the centre of gravity of the load acting on the powered supports can migrate rapidly to the front of the supports, with the resulting moment arm resulting in a rapid and significant reduction in the total load carrying capacity of the powered supports and a step increase in floor pressure under the toes of the powered support bases.
- There is a loss of face height, resulting from some or all of the above factors.
- Stability and face advance are highly dependent on the longwall face holing into the pre-driven roadway at or very close to the same floor and roof elevation and this can be difficult to achieve, especially in the presence of floor heave, seam convergence and roof instability.
- The consequences of any downtime are higher, given that the strength of stressed rock, particularly sedimentary rock, can be time dependent.
- There is a higher likelihood of unplanned downtime due to the impact of many of the preceding factors on face operations, with the highest probability of downtime coinciding with the critical stage of holing through when the consequences of downtime are highest.

Impacts commonly associated with these factors are:

- roof falls on the longwall face due to the increased tip-to-face distance;
- a face break, which then results in the fender and powered supports being loaded, often rapidly, by a detached block;
- dynamic and violent failure of the fender, sufficient in one instance to have caused serious damage to hydraulic circuitry on the powered supports;
- loss of horizon control and clearance for the shearer on the longwall face due to the AFC being lifted and tilted by floor heave;
- inability to advance the powered supports due to a difference in floor or roof horizon between the pre-driven roadway and the longwall face;
- inability to generate tip support due to a difference in roof horizon between the pre-driven roadway and the longwall face;
- reduce powered support capacity due to damage to hydraulic circuitry; and
- trapped and iron bound equipment.

Case studies provide insight into strata behaviour around pre-driven roadways and the factors that influence success. Simpson et al. (1991) report on a series of successful pre-driven roadways in three different seams at Newstan Colliery, Australia. The powered supports were 4 leg chock shields, face width ranged from 118 to 201 m, extracted height from 3.0 to 3.4 m, and depth from 20 to 90 m, except for one case at a depth of 300 m. The immediate roof of each of the pre-driven roadways was supported by various combinations of rock bolts, straps, mesh, 10 m long fully encapsulated cables, and monolithic cylinders. The outbye riblines were supported with 1.8 m long steel bolts and mesh and the inbye riblines with various cuttable bolts and dowels (up to 8 m in length) and strata binders. The face operation mode reverted to conventional (effectively, double chocked) as the pre-driven roadway was approached, with the maingate leading by up to 8 m.

Floor heave occurred in all instances, with up to 1 m occurring within one hour in the two shallowest seams. It is reported that this was easily cut and loaded out by the shearer.

Extensive monitoring of the pre-driven roadway developed at a depth of 300 m revealed that although there was a rapid increase in roof to floor convergence as the fender width was reduced from 8 to 4 m, there was no bed separation within the first 11 m of the immediate roof. It was only during the last 4 m of extraction when the fender failed that the immediate roof developed partings up to the 4 m horizon. The controlled nature of the fender failure in the shallower seams was believed to be associated with the bearing failure of the floor. However, this could not account for controlled failure in the deeper seam where the floor was a 100 MPa strength shale.

The behaviour of the immediate roof at Newstan Colliery contrasts with that associated with the unsuccessful cases shown in Fig. 9.25a, c, d. The Longwall Panel 6 failure in 1987 at Pacific Colliery, Australia, depicted in Fig. 9.25a, was associated with a 138.5 m wide panel being extracted at a height of 2.6–2.7 m utilising 2 leg, 5.6 MN (560 t) capacity, shield supports. The immediate roof comprised weak tuffaceous sediments (claystone) overlain by stronger mudstone and sandstone strata. The panel had been subject to periodic weighting that was attributed to the presence of the sandstone. The pre-driven roadway was supported by a combination of 2.4 and 2.7 m long fully resin encapsulated bolts installed through straps, and 8 and 10 m long cementitious grouted cables. No standing support was installed in the recovery roadway.

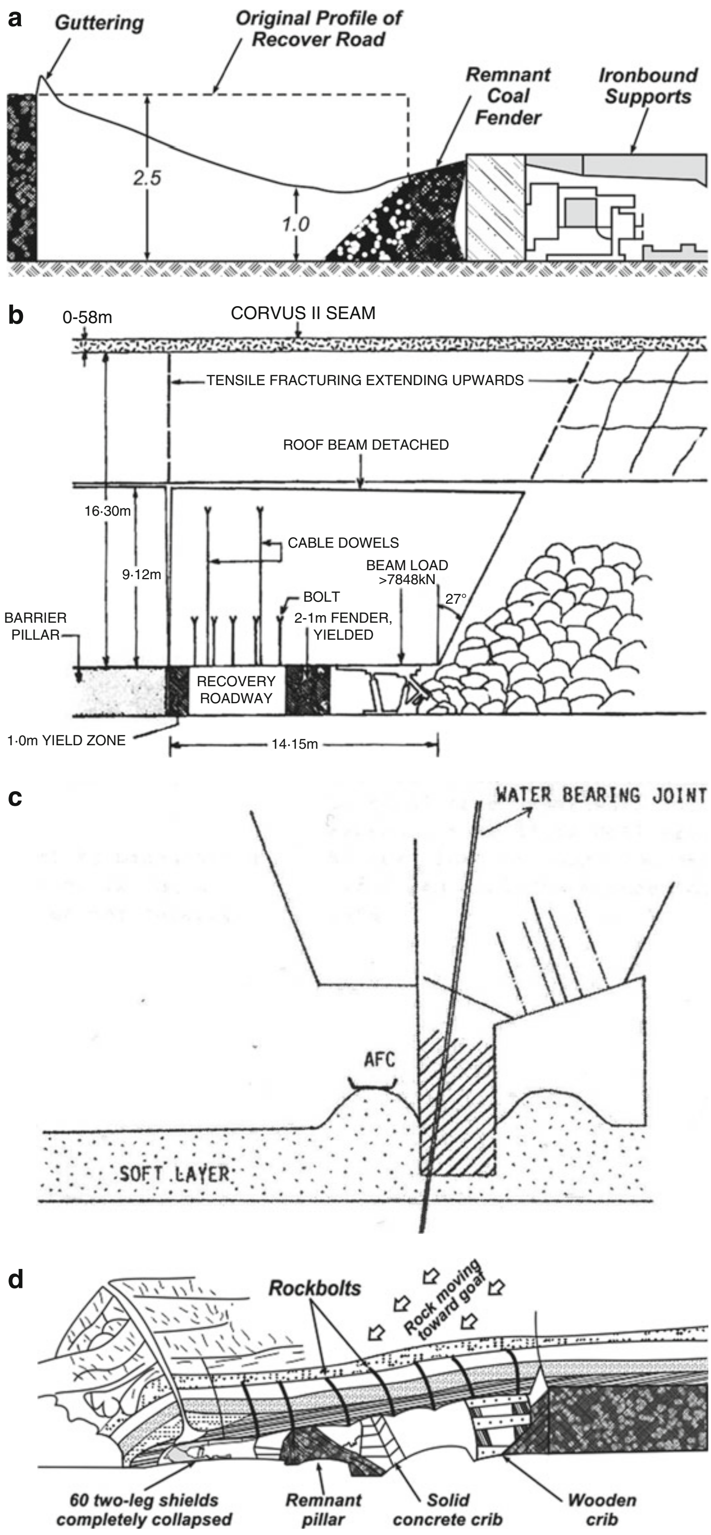
Salient points associated with this failure as described by Gardner (1987) are:

- The ranging arm on the shearer failed when the face was some 22 m away from the recovery roadway, resulting in a 16 h stoppage.
- The fender yielded at a width of 7 m, generating an 18 m long cantilever.
- 1:30 am, 19/7/87: Fender width 3 m. W straps in recovery road roof started to buckle, and small amounts of guttering were apparent in outbye corner of the roadway.
- 5:00 am: Powered supports along face went onto continuous yield, at the same time as the roof in the recovery roadway underwent *'dramatic deterioration with much fretting between W-straps and around cable bolt plates'*.
- 7:00 am: Upon the shearer holing the maingate end of the recovery road, it was found that the roof of the recovery roadway was much lower than the longwall face. The powered supports still had about 150 mm of leg travel remaining. *'At this time the remaining coal fender, about 2 m thick, literally exploded into the recovery roadway.'*
- 7:30 am: Shearer was unable to pass under support canopies. Nearly all legs had less than 50 mm of travel remaining.
- 9:30 am: All roof movement appeared to have ceased. Powered supports were now iron bound. Most of the recovery roadway was still intact but in very poor condition. During the next 6–8 h, some sections did fall.

One of the causes of the incident proposed at the time was the failure of a sandstone cantilever ahead of the face that resulted in the powered supports being loaded well in excess of their capacity. This mechanism is similar to that concluded from instrumentation by Klenowski et al. (1990) in regard to difficulties experienced in holing into the pre-developed recovery roadway for Longwall Panel 303 at Central Colliery in 1990 (Fig. 9.25b). Salient points associated with this experience were:

- The pre-driven recovery roadway was supported with bolts and straps that were subsequently augmented with 8 m cables in the roof and dowels in the fender.
- The support design was based mainly on the performance of an instrumented stub heading which extended 15 m into the panel, complemented with finite element modelling. The stub fender yielded at a width of 2.7 m.
- The pre-driven roadway fender yielded at a width of 5 m. However, it did not fail until there was one shear remaining, when maximum recorded face convergence was almost 260 mm.

**Fig. 9.25** Cross-sections through three unstable (**a**, **c**, **d**) and one troublesome (**b**) pre-driven longwall recovery roadways. (**a**) Pacific Colliery, NSW, Australia, 1987 (After L. Gardner 1987), (**b**) Central Colliery, Qld, Australia (After Klenowski et al. 1990), (**c**) Sasol Colliery, South Africa (After Van der Merwe 1989) (**d**) Colorado, USA (Adapted from Oyler et al. 1999, from Pulse 1990)


a
Guttering
Original Profile of Recover Road
Remnant Coal Fender
Ironbound Supports
2.5
1.0
b
0-58m
CORVUS II SEAM
TENSILE FRACTURING EXTENDING UPWARDS
ROOF BEAM DETACHED
16·30m
9·12m
CABLE DOWELS
BEAM LOAD >7848kN
27°
BOLT
2-1m FENDER, YIELDED
BARRIER PILLAR
RECOVERY ROADWAY
1·0m YIELD ZONE
14·15m
c
WATER BEARING JOINT
AFC
SOFT LAYER
d
Rockbolts
Rock moving toward goaf
60 two-leg shields completely collapsed
Remnant pillar
Solid concrete crib
Wooden crib

- Convergence increased to at least 420 mm at the shearer as the last shear was being taken.
- At the time of holing in, guttering had developed along the outbye ribline over a 10 m distance at mid-face.
- Convergence was greater on the longwall face side of the fender than the outbye side, requiring the setting height of the powered supports to be increased immediately after holing. Insufficient hydraulic fluid was available for this to occur and so powered support advance ceased until the pumps reservoirs were refilled. It appears that gas yield valves commenced to malfunction at this time.
- Back-analysis indicated that a 9.3 m thick roof beam with an average length of some 21 m detached from the outbye side of the recovery roadway and commenced to crush the fender when it was 5 m wide. Virtually the full weight of the beam was taken by the powered supports. At a fender width of 2.1 m, the height of the detached beam increased, with its pivot point being over the outbye pillar. At the completion of holing in, a second roof beam located 16.3 m above the seam had also detached. More than 104 powered support legs were defective by that stage and four timber props had to be set under the canopy of each support.

It was concluded at the time that roof-to-floor convergence continued and timber props progressively failed because the total resistance provided by the functional support legs and timber props was 10.1 MN (1,010 t) per support, compared to a beam load of 10.68 MN (1,068 t). Greater insight can be gained today by considering where the centre of gravity of this load acted and the need to also balance moments in order to maintain stability. Application of the detached model presented in Sect. 9.3.3 and defined by Eq. 9.2 indicates that the maximum load capacity of the powered supports for the geometry depicted in Fig. 9.25b was only of the order of 950 kN (95 t). Hence, aggressive support yield was inevitable. The front legs of the powered supports would have had to have a combined load carrying capacity in excess of 43 MN (4300 t) to avoid the support going into yield. Based on static analysis, the maximum thickness of detached block that could have been sustained without the powered supports going into yield was only about 2 m.

Van der Merwe (1989) reported on an incident in South Africa in which the conveyor belt tore when the face was only 3 m from holing. The repair took 8 h, during which time the face pillar (fender) slowly punched some 1.5 m into the floor. The floor heave lifted the AFC, which in turn, lifted the shearer so high that it could not pass beneath the powered supports (Fig. 9.25c).

Figure 9.25d shows a cross-section through a failed pre-developed roadway in Colorado, USA, as reported by Oyler et al. (1999) and attributed to Pulse (1990). The fender was between one and two metres thick when the AFC pan line became stuck, causing face advance to stop for 6 h. The mechanics of the detached block are similar to those shown in Fig. 9.25a, b and to a failure in Australia in 2011, shown in Fig. 9.26, where the fender and the standing support in the pre-driven roadway punched into the floor under the deadweight of a detached block.

Hanson et al. (2014) report on two successful pre-driven longwall recovery roadways at a depth of 70 m at Bull Mountains Mine No. 1 in the USA. The first pre-driven roadway was some 10 m wide and supported with a combination of rock bolts, cable bolts and cuttable and non-cuttable cementitious cribs. Problems installing the recovery mesh over the longwall powered supports caused extended delays which allowed the roof to deteriorate. During this time, there was significant load transfer to the cribs and powered supports.

The second pre-driven roadway was almost 13 m wide and driven in two 6.5 m wide passes. The first pass was supported with bolts, steel mesh and 5 and 8 m long cables of 55 tonne capacity, before then installing the longwall recovery mesh. Next, the roadway was completely backfilled with a cuttable, low-density, 5.5 MPa concrete. After the concrete had cured the outbye second pass was driven, bolted, meshed and also completely backfilled. Full contact of the concrete with the roof was verified, with voids being filled with a 5.5 MPa

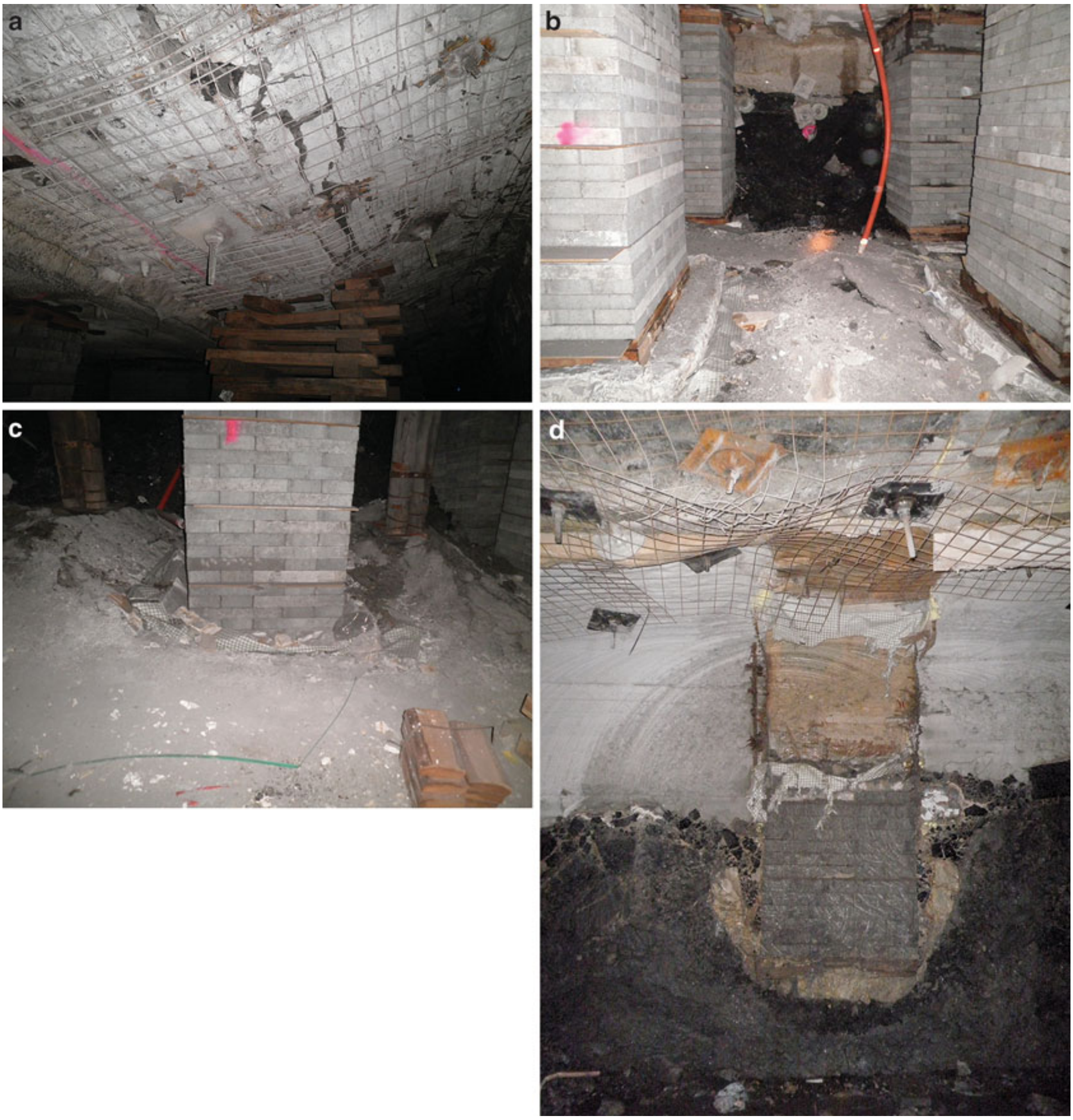


**Fig. 9.26** Failure of a pre-developed roadway in 2011 at an Australian colliery, associated with punching of fender and standing supports into soft and weak floor under deadweight load of a detached block. (**a**) Pivot point of detached roof block close to edge of outbye pillar. (**b**) Standing support starting to punch floor. (**c**) Floor heaving as bearing failure occurs beneath standing support. (**d**) Extent of punching of standing support into floor strata

polyurethane. When the fender pillar was 3 m wide, load transfer occurred onto the backfill and the longwall powered supports, with the shearer having no trouble cutting through both the coal and the concrete. An approximately 150 mm thick layer of concrete was left against the roof and this peeled off easily after being undercut to expose the pre-installed roof support and permit the recovery mesh to lay down on the shields.

Experience demonstrates that the risk of ground instability associated with excessive tip-to-face distance is elevated significantly when driving into any pre-driven roadway. This risk is magnified significantly if a detached block develops in the roof during this process. Static analysis highlights that moments, rather than forces, are the dominant factor determining stability in these situations. Hence, the capacity of the powered supports has limited influence on the outcome.

Instability is initiated by failure of the fender pillar system, either through bearing capacity failure of the fender foundations or yielding of the coal seam element. The consequences of this failure are determined by the stiffness of the loading system relative to the stiffness of the coal pillar system. As the stiffness of the surrounding strata is a function of elastic modulus and span, the probability of rapid loading leading to dynamic failure is reduced in the presence of more competent roof strata, provided that a face break does not develop towards the outbye rib of the pre-developed roadway. A face break in any type of strata immediately reduces the stiffness of the detached strata to zero, resulting in a high likelihood of uncontrolled failure when the strength of the fender pillar system is exceeded. As periodic weighting is prone to produce face breaks, there is an elevated risk of failure in periodic weighting environments.

A consideration of moments highlights that a detached block of only a few metres in thickness is sufficient to cause yielding of powered supports. Tendon reinforcement systems have limited influence on the development of a detached block, both because their capacity is insufficient to resist the forces generated by the turning moment of the detached block and because the height of the detached block extends beyond the reach of the tendons.

Panel span and distance from the panel corners impacts on the stiffness of the surrounding strata and, therefore, on the vertical and lateral extent of a detached block and the rate of loading of the fender system. Hence, limited reliance should be based on trial excavations that are of restricted extent and/or located to one side of a panel.

Given the complex combination of factors that contribute to behaviour when holing into a pre-driven roadway and the opportunities and threats associated with this practice, it is important that design is premised on an applied mechanics approach and that this is underpinned with a good understanding of the surrounding geology, material properties and stress environment. The insight provided by appropriate numerical modelling is illustrated by studies such as those of Tadolini and Barczak (2004) and Zhang et al. (2006). Tadolini and Barczak (2004) utilised a calibrated three-dimensional finite element model, developed in conjunction with an underground test site, to undertake a parametric study of the critical components and design principles relevant to a pre-driven roadway. The modelling identified critical stress distributions, failure modes, and failure locations which then provided a basis for selecting support types, patterns and densities and for anticipating critical events such as yielding of the fender.

Experience supported by numerical modelling highlights that careful consideration needs to be given to the following when planning to mine into a pre-driven longwall recovery roadway:

- selecting standing support with appropriate stiffness, strength and yielding properties, and erecting it to a pattern and density such that it is capable of sustaining both convergence and the deadweight load of a detached block without disintegrating or punching the roof or floor strata;
- reinforcing and suspending the immediate roof of the pre-driven roadway with long cables angled over both the outbye rib and the fender as well as installed vertically;
- binding the immediate roof together with bolts and mesh to provide confinement to the overlying strata and to prevent unravelling and local roof falls;
- reinforcing the fender with cuttable tendons and the outbye rib with steel tendons;
- ensuring a high level of integrity of the hydraulic circuitry for powered supports;
- consistent and rapid rate of retreat which, in turn, relies on addressing a range of other factors such as coal clearance capacity, maintenance schedules and labour availability;
- operation of the longwall face in a closed up state (doubled chocked);
- recognition that the fender may fail in a sudden and violent manner and implementing controls to safely manage such a situation;
- driving the face into the pre-driven roadway at an angle to avoid weakening the whole face in one pass and to reduce the likelihood of pillar failure rapidly developing along the full length of the face; and
- maintaining horizon control such that the longwall face holes in at the same roof and floor elevation as the pre-driven roadway.

Backfilling of a pre-driven roadway tightly to the roof with a suitably stiff material can reduce or eliminate the risk associated with some of these factors.

## 9.8 Longwall Face Recovery

The relocation of a longwall face presents a number of ground control challenges, particularly because the powered supports have to be extricated from a goaf in circumstances where the tip-to-face distance has been deliberately increased to the order of 2.5 m to provide sufficient working space and because the strata has time to deteriorate due to the duration of the process. Operations may also have to contend with factors that are unknown prior to the commencement of recovery, such as geological features and the state of loading in a periodic weighting cycle. Success is contingent on safe, speedy recovery of face equipment which, in turn, is very dependent on the standard of preparation of the recovery face and the roadways leading to it; monitoring of strata deterioration in these excavations during recovery operations and implementation of timely and effective responses; and powered support withdrawal procedures. These should be elements of a comprehensive risk assessment of all aspects associated with relocating a longwall face.

Basically, face equipment may be recovered via one or both gate ends and/or from roadways, or **chutes**, that hole into the recovery roadway at various points along its length (Fig. 9.27). Sometimes, the first 20–50 m of one or both ends of the recovery road may be pre-driven. Stability of intersections on the recovery face line is critical, particularly at the maingate end if the face is recovered predominantly through that access point. Cable bolting of these areas should be a matter of routine.

As the longwall approaches the pull-off point, the immediate roof needs to be treated in order to control goaf flushing during the extraction of the powered supports. This is usually achieved by commencing to screen and rock bolt the roof when the longwall approaches within 10–15 m of its final position. The screen often comprises polyester geotextile that has been stitched together before being taken underground so as to form one continuous sheet that ultimately extends out from under the goaf pile and over the top of the powered supports and partially down the face (Fig. 9.28). Pattern bolting of the roof should extend back to at least the rear of the powered supports, typically at no more than 1 m

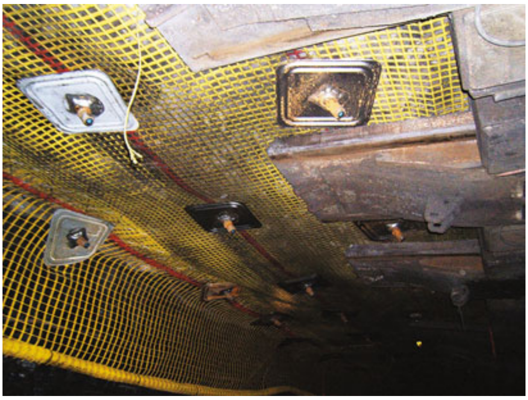


**Fig. 9.28** A longwall recovery roadway that has been supported with polyester screen and bolts

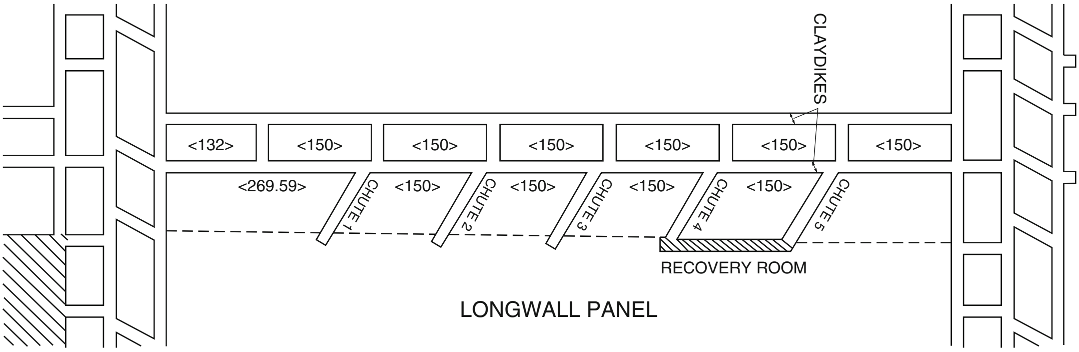


**Fig. 9.27** An example of longwall recovery roadway layout utilising chutes (After Tadolini and Barczak 2004)

centres over the supports. In the tip-to-face region, bolt density may exceed one bolt per square metre. If these bolts are made from high tensile steel, caution is required that the bolt tails do not become projectiles if sheared off by the canopies of the powered supports as they are advanced. Some mines specify mild steel bolts for longwall face bolt up to mitigate this stored energy risk. In shallow mines, the recovery area may be pre-supported by installing cable bolts from the surface (reference, for example, R. Butcher and Kirsten 1999).

In weak to moderate strength roof strata environments, consideration should be given to installing one to two rows of cable bolts along the final face recovery position, with the face-side row angled over the ribline. It is advisable that cable bolts be at least 6 m long, with longer and higher capacity bolts recommended in areas affected by periodic weighting and geological features.

As mine personnel and equipment are exposed to the coal face in a confined space in circumstances where the ribline is subjected to abutment stress over an extended period of time, it is essential that the coal face is supported. Extending the roof screen down the ribline and pinning it with rib bolts is one commonly employed method (Fig. 9.29). Payne (2008) reported that at Crinum Mine, Australia, a flexible rib spray product in combination with friction bolts worked reasonably well in conditions where face spall had been a problem when using synthetic grid mesh and resin anchored rib bolts.

The capacity of the floor strata to sustain abutment stress, horizontal stress, and high, repetitive vehicle axle loads is another important geotechnical consideration when planning a longwall recovery. Deterioration in floor conditions can directly and indirectly lead to delays in recovering equipment when, from a geotechnical perspective, time may be of the essence. Therefore, a roadway maintenance scheme that includes provision for concreting poor sections of roadways prior to commencing to recover the longwall is advisable.

Having designed and implemented support systems in and about the longwall recovery face, these need to be complemented with monitoring instrumentation and timely data processing procedures in order to detect any deviations from as-designed and to underpin a robust Trigger Action Response Plan. Commencement of recovery of longwall face equipment should be commensurate on all of the preceding measures being in place. Ground control problems during the extrication of the powered supports usually arise from one or a combination of goaf flushing into the working area; roof convergence in and about the recovery site; physical interference between supports during extraction; and roof deterioration along the egress path.

The manner and sequence in which the powered supports are extricated from the goaf can vary, depending on factors such as ventilation, number of take-off points, type of recovery equipment, ground conditions, and room to manoeuvre. Within a recovery length, supports

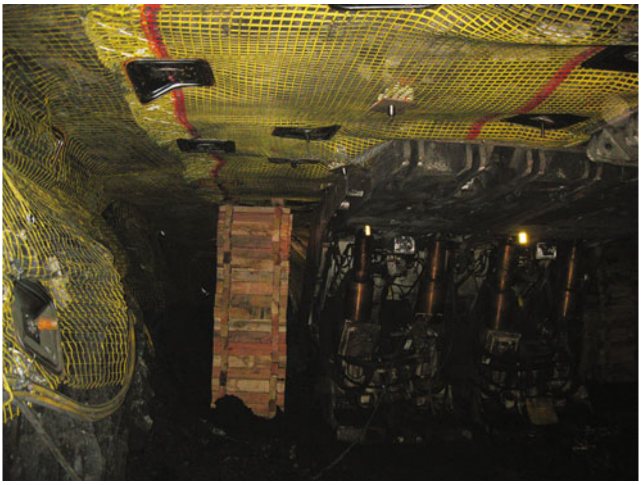


**Fig. 9.29** Looking inbye on a longwall recovery roadway showing meshed coal face and roof, rock bolts and cable bolts, walker (buttress) chocks and Link-n-Lock® timber chocks to maintain a return airway

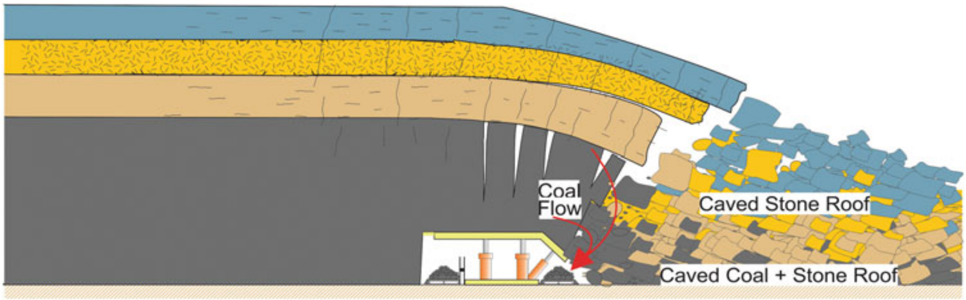


**Fig. 9.30** A conceptual vertical cross-section through a LTCC face

may be recovered in a linear or alternate sequence. It is common to set two or three powered supports longitudinally to the coal face at the site of support recovery in order to provide increased protection from goaf flush and to act as a break off point for goaf falls (Fig. 9.29). These are referred to variously as **buttress chocks**, **walker chocks** or **walking shields** and sometimes comprise mobile roof supports (MRS). If a return airway has to be kept open along the recovery face, timber chocks and props may also need to be installed. More in-depth appraisals of the geotechnical planning and design to support longwall face recovery operations are provided by Hill (2006, 2010).

## 9.9 Other Longwall Variants

### 9.9.1 Longwall Top Coal Caving

**Longwall Top Coal Caving (LTCC)** involves longwall mining the lower portion of a thick coal seam and drawing off the upper portion as it caves into the goaf (Fig. 9.30). The method evolved in Europe, where it was known as **soutirage** or **integrated longwall mining with sublevel caving**, and incorporated either a chute in the canopy of each powered support to direct caving coal on the face conveyor or a retractable gate in the rear shield plates to control coal flow onto a second conveyor at the rear of the supports (Fig. 9.31). In many early operations, a second conventional longwall face extracted a slice from the top of the thick seam at least one month in advance of LTCC operations, in a method known as **non-integrated longwall mining with sub-level caving**.

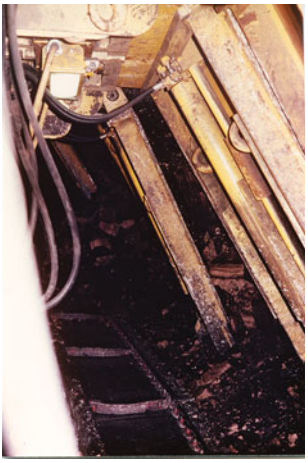


**Fig. 9.31** A Hemscheidt LTCC face with gates incorporated into the shield plates, operating at Velenje, Slovenia (After Galvin 1978)

The concept of LTCC was introduced into China in 1982 (Cai et al. 2004). The Chinese made significant improvements to the rear loading technology by installing supplementary, hydraulically activated, shield plates behind the supports to protect the rear conveyor, to enable it to be retracted independently of the front conveyor, and to provide better control over the drawing of coal. An example of such a support is shown in Fig. 9.32. The method now finds application to seam thicknesses typically ranging from 4.5 to 12 m, with the upper extraction height being constrained by geotechnical considerations and the reach distance of the rear conveyor into the goaf to recover coal flow from greater heights.

A considerable amount of the early research into LTCC was undertaken in France in the 1960s and 1970s and produced the model of coal flow and corresponding displacements shown in Fig. 9.33 (Adams 1976). A range of

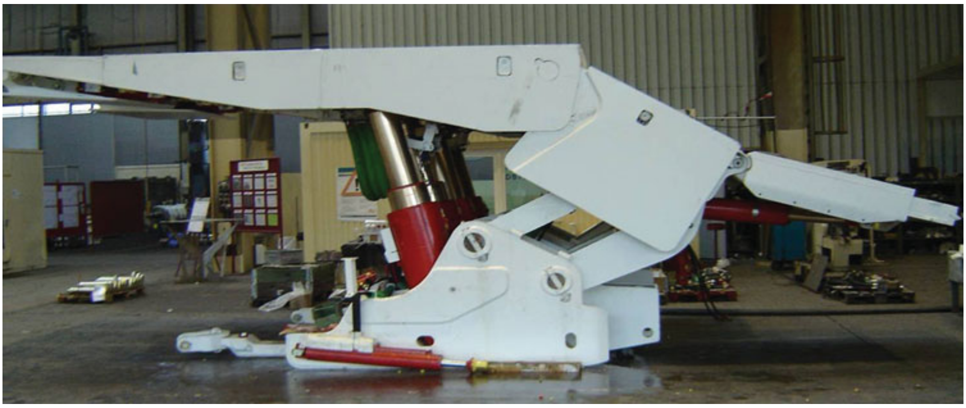


**Fig. 9.32** A modern Chinese LTCC powered support

investigations at Chinese mines has produced similar generic conclusions.

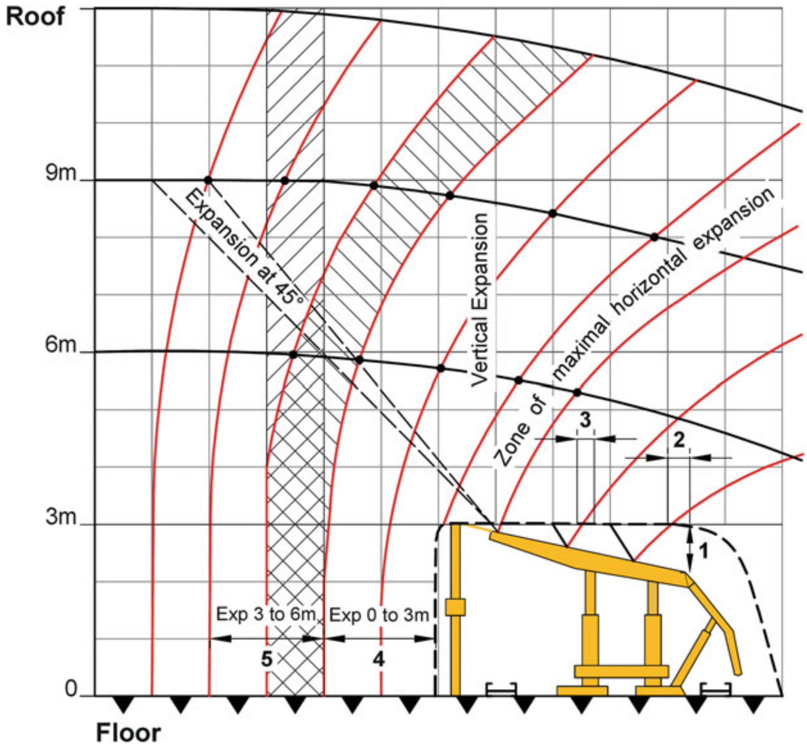
The success of LTCC lies in the friability and caveability of the coal, which is a function of coal material strength; cover depth; structural inclusions (joints, cleat etc.); stone bands within the top coal; and the nature of the overlying roof strata. Depth, as encapsulated in face abutment stress, provides the driving force to fracture the coal. The lack of abutment stress when mining beneath the goaf of an upper slice is one of the main reasons why this version of the technique is no longer utilised. In-seam bands can impede caving and block draw points.

The nature of the immediate stone roof determines the frequency and area of caving. LTCC effectively extracts the parting and removes the cushioning to subsiding upper strata. Risks of windblast and equipment damage arise if caving of this material does not follow closely behind face advance. Delayed caving of the immediate and/or upper stone roof also gives rise to the risk of periodic weighting. Moodie and Anderson (2011) reported that during periodic weighting episodes at Austar Coal Mine, Australia, caving operations were temporarily suspended and the speed of retreat was increased. This resulted in less intense loading events spread over a longer period of retreat. The authors also reported that a stress rotation occurred in the tailgate, consistent with the skew roof model of Tarrant (2005a), which may be exacerbated by the increased extraction associated with LTCC.

Caving must be restricted towards the ends of the longwall face to maintain the integrity of the gate ends, interpanel pillars and structures installed within cut-throughs, such as ventilation stoppings and seals. It must also not be permitted to approach or over-run the face, with removal of lateral confinement being conducive to face falls. The gate ends require very careful management from a local ground control perspective as the second conveyor is lagging in the goaf and persons require safe access to this equipment.

### 9.9.2 Miniwall

A **miniwall** is a longwall mining variant in which a single ranging arm shearer works to a blind end, with ventilation being returned over the goaf. Face length is restricted to the order of 50 m and two panels may be extracted from the one set of main developments. The method has found application in shallow situations in Australia where panel width has had to be restricted in order to limit surface subsidence. Its success is dependent on a strong immediate roof strata which does not cave to the extent of choking off return ventilation over the goaf and does not present a windblast risk if caving is delayed. More detailed information is to be found in McKendry and Simes (1987), Simes (1989) and Hedley and McDonald (1993).


Roof
9m
6m
3m
0
Floor
Expansion at 45°
Vertical Expansion
Zone of maximal horizontal expansion
3
2
1
Exp 3 to 6m
Exp 0 to 3m
5
4

| Characteristics | | | | | | Results of Measurements | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Mines** | **Face** | **Seams** | **Depth (m)** | **Thickness (m)** | **Cutting Height (m)** | **1** | **2** | **3** | **4** | **5** | **Expansion Total %** |
| **Darcy** | Taille D | 4th | 800 | 12.3 | 2.8 | 1155 | 254 | 120 | 0.38 | 3.35 | 20 |
| **Rozelay** | T.3b | 2nd | 310 | 12 | 2.4 | 660 | 160 | 38 | | 2.1 | 9 |
| **Ricard** | Taille 2 | 1st | 840 | 4.5 | 2.5 | 587 | 131 | | 0 | 0.5 | |

Notes: Numbers in the Figure are the locations where the measurements were taken:

1. Convergence mm;
2. Convergence per metre of face advance;
3. Displacement of roof beams between setting and advancing;
4. Expansion 3 to 6m;
5. Expansion 0 to 3m

**Fig. 9.33** Outcomes of research undertaken in France in the 1960s and 1970s into coal flow associated with LTCC (After Adams 1976)

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# 10

# Overburden Subsidence

**Abstract**

The term subsidence has two meanings in ground engineering. It can encapsulate all mining-induced movements of the overburden and the ground surface above, as it does in this text, or it can refer specifically to vertical displacement of the ground. In this chapter, the significance of subsidence is discussed in terms of effects, impacts, and consequences, with these terms ascribed the following meanings:

- Effect – the nature of a particular mining-induced ground movement.
- Impact – any physical change to the fabric of the ground, its surface, or a man-made feature resulting from a subsidence effect.
- Consequence – any change in the amenity, function or risk profile of a natural or man-made feature due to a subsidence impact.

Subsidence behaviour associated with secondary extraction systems is classified under one of three headings, recognising that in practice there will be transitional states between these scenarios. Consideration is then given to the behaviour of the superincumbent strata between the mining horizon and the surface and to a number of models that have been proposed to represent this behaviour. This forms the basis for considering subsurface effects and impacts, including those on groundwater.

Consideration is then given to subsidence effects, impacts and consequences at the surface. Subsidence is classified under the headings of Sinkhole and Plug Subsidence; Classical Surface Subsidence; and Site-centric Subsidence. Research findings in relation to Valley Closure, Upsidence and Far-field Movements are presented. Prediction methodologies are discussed along with subsidence impacts and consequences and measures to mitigate and remediate these. Photographs are used to illustrate a variety of related field experiences.

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**Keywords**
Angle of draw • Backfill • Bulk porosity • Caved zone • Chimney cave • Closure • Constrained zone • Curvature • Deflection ratio • Far-field movement • Fractured zone • Groundwater • Horizontal displacement • Hydraulic conductivity • Permeability • Plug failure • Point of inflexion • Pothole • Protective pillar • Remediation • Shear strain • Sinkhole • Storativity • Strain • Subsidence • Subsidence factor • Subsidence models • Sub-surface subsidence • Surface subsidence • Surface watercourses • Tilt • Underflow • Uplift • Upsidence • Valley bulging • Valley closure • Vertical displacement

## 10.1 Introduction

The term **subsidence** has two meanings in ground engineering. It can encapsulate all mining-induced movements of the overburden and the surface, as it does in this text, or it can refer specifically to vertical displacement of the ground. In this chapter, the significance of subsidence is discussed in terms of **effects**, **impacts,** and **consequences**, with these terms ascribed the following meanings:

- Effect – the nature of a particular mining-induced ground movement.
- Impact – any physical change to the fabric of the ground, its surface, or a man-made feature resulting from a subsidence effect.
- Consequence – any change in the amenity, function or risk profile of a natural or man-made feature due to a subsidence impact.

Some of the first systematic investigations into mine subsidence were conducted in Belgium in the early 1820s as a result of widespread surface movements and damage to buildings above coal mine workings in the city of Liege, Belgium (Shadbolt 1977). Two commissions of inquiry followed, one in 1825 and the second in 1839, and by 1880, a number of empirically based theories to account for vertical displacement above mine workings had been developed in Belgium, Germany, France, and Great Britain. This work provided the theoretical basis of what is termed 'classical' subsidence behaviour for the purpose of this text, founded on the assumption that both the surface topography and the mining horizon are level.

In the early 1990s, it started to be recognised from field experience in Australia that classical subsidence theory did not account for all observed subsidence effects and impacts at the surface. These deficiencies were most apparent when mining beneath steep or deeply incised topography and in high in situ lateral stress settings. Newly identified effects included far-field horizontal movements, valley closure and valley floor uplift. Subsequently, these effects have been associated variously with so-called 'non-systematic', 'unconventional' or 'disordered' subsidence (reference, for example, DoP 2008).

None of these terms are ideal as there is nothing non-systematic or unconventional about the behaviours; they are predictable physical responses to the formation of excavations in a rock mass. Rather, it is just that the behaviours are more pronounced in specific settings. In this text, therefore, non-systematic, unconventional or disordered subsidence is referred to as **site-centric subsidence behaviour**, while so-called 'systematic', 'conventional' or 'ordered' subsidence is referred to as **classical subsidence behaviour**.

## 10.2 Generic Behaviours

Mechanisation and advances in mining engineering and ground engineering result in most coal mining layouts now comprising a regular array of excavations and pillars. Furthermore, secondary extraction systems are associated with excavations that are very long relative to their width. Therefore, behaviour across a series of panels can be visualised and simulated reasonably accurately in two-dimensions provided that the area of interest is more than 1–1.5 times the panel width from the start and finish ends of the panels. Subsidence in these circumstances can be discussed on the basis of three, two-dimensional, generic mining layouts, recognising that in practice there will be transitional states between these scenarios. The three subsidence situations evolve out of the principles pertaining to the behaviour of excavations (Chap. 3) and pillar systems (Chap. 4), with aspects of them already having been discussed in regard to pillar extraction (Chap. 8) and longwall mining (Chap. 9). They are:

- Critical and supercritical panel width-to-depth ratio, W/H, as defined in Sect. 3.3.1 and shown in Fig. 10.1a. The vertical displacement profile depicted in this diagram is typical for total extraction panels at depths of less than 200 m, as reflected in Fig. 3.15. Subsidence behaviour over one panel is largely independent of that over adjacent panels. This is because the panel width is sufficiently large relative to the depth of mining to result in the extraction of one panel reducing the stiffness of the superincumbent strata to almost zero.
- Subcritical panel width-to-depth ratio, W/H, at considerable depth, shown in Fig. 10.1b. In this case, the panel width has not changed from that shown in Fig. 10.1a but the depth of mining has increased so that extraction of one panel is no longer adequate to reduce the stiffness of the superincumbent strata to zero. Several panels may need to be extracted for this to occur, resulting in interaction between excavations. This is typical of deeper situations (say, >300 m) where excavation width is less than depth of mining and overburden loads are high, such as for the case shown in Fig. 3.16.
- Subcritical panel width-to-depth ratio, W/H, at shallow depth, shown in Fig. 10.1c. The depth of mining has not changed from that shown in Fig. 10.1a but panel width is considerably less. This is typical of first workings and of partial extraction second workings designed to restrict subsidence. Both superincumbent stiffness and pillar stiffness are high in order to restrict subsidence and, therefore, increasing the overall extent of mining, $W_o$, has little effect on the system stiffness of adjacent workings.

Vertical displacement at the surface can be conceptualised as comprising two components, being sag of the superincumbent strata; and compression of excavation abutments (including interpanel pillars) under the weight of that superincumbent strata not supported by the goaf. Sag and abutment compression are a function of:

- The caving, fracturing and compaction characteristics of the superincumbent strata, with these determining how the weight of the undermined superincumbent strata is apportioned between the panel abutments and the goaf.
- The pre-failure and the post-failure stiffness of the interpanel pillars and the strata above and below them.
- The stiffness of the solid abutments flanking a series of excavations. Because these abutments are usually very wide, they are stiffer and compress less under abutment load than the strata associated with an interpanel pillar.

These components and their interaction in determining system stiffness and, therefore, vertical displacement at the surface, can be conceptualised by a system of coil springs of various stiffnesses and a plate spring as illustrated in Fig. 10.2.

## 10.3 Sub-surface Subsidence

### 10.3.1 Fundamentals

Historically, sub-surface subsidence behaviour has been the domain of mining engineers and strata control practitioners whose interest has been primarily in respect of safety and asset protection associated with loss of water from surface storage impoundments; dynamic inrush into mine workings of fluids (principally water) and materials that flow when wet; inundation (flooding) of mine workings over time; and interaction between multi-seam workings. However, it has now come under greater focus due to the increased importance attached to the environmental and community impacts and

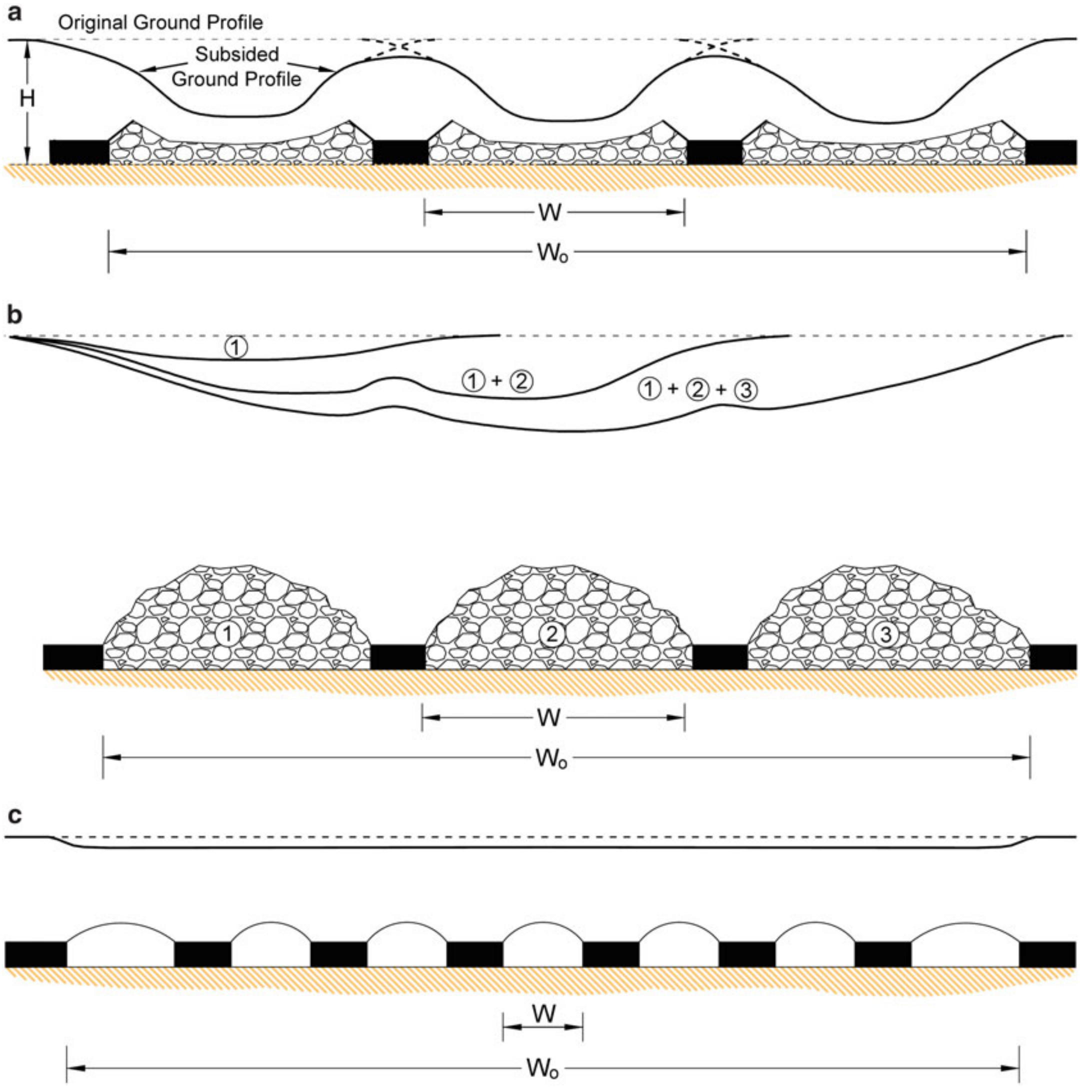


**Fig. 10.1** Two-dimensional visualisations of generic secondary extraction layouts pertaining to underground coal mining and the vertical displacement surface profiles associated with each layout. (**a**) Individual panel width-to-depth ratio, W/H, critical or supercritical. (**b**) Individual panel width-to-depth ratio, W/H, subcritical in a deep mining situation. (**c**) Individual panel width-to-depth ratio, W/H, subcritical in a shallow mining situation

consequences associated with subsidence of groundwater and surface water systems. Hence, the topic has also become the domain of other disciplines, most notably hydrogeologists and environmental engineers.

Differences in the evolution and interests of these various professions have given rise to a variety of technical perspectives and confusion concerning some terminology. This text is concerned primarily with presenting the basic mechanisms associated with subsurface subsidence and with providing a generic understanding of how these affect surface and groundwater systems. More detailed discussions of hydrogeological processes is provided in publications such as Booth (2002).

Mining can affect groundwater and surface water flows through two basic mechanisms, referred to simplistically as **drainage** and as **subsidence fracturing**. In all situations, mining creates the potential for groundwater flow to be diverted to a depressurised void, or sink. In the

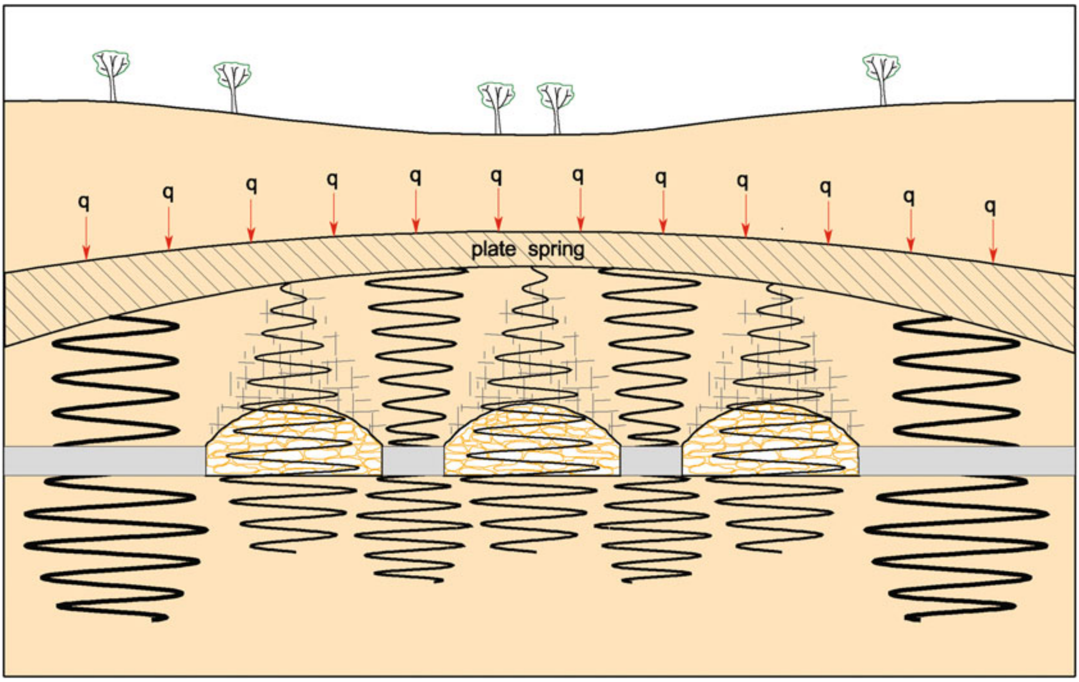


**Fig. 10.2** Conceptualisation of how vertical displacement at the surface is a function of the stiffness of the superincumbent strata, simulated by a plate spring, and the stiffnesses of the goaves and various pillar systems, simulated by coil springs

absence of mining-induced fracturing, drainage (water inflow) to this sink is determined by the natural permeability of the surrounding strata and the pressure head of the water within this strata. In some instances, mine workings may remain dry in the very long term, while in other cases the rate of seepage into the sink may be considerable. Subsidence fracturing, on the other hand, is associated with an increase in permeability and/or storage capacity of strata as a result of mining-induced fracturing, opening of natural joints, bed separation and caving. The impacts of both mechanisms are influenced significantly by the nature and distribution of low permeability strata units and may include regional drawdown of some or all aquifers.

The volume of water stored in a rock mass is described by its **bulk porosity**. Groundwater flow occurs in connected pore spaces within the rock matrix and in fracture networks within strata units. The rate at which a fluid can travel through a rock matrix or through a fracture network is governed by the **permeability**, k, of the system. Permeability is often loosely associated with the term **hydraulic conductivity**, K. However, as shown by Eq. 10.1 the two terms differ, as do their units of measurement.

$$k = \frac{K\mu_d}{\rho g} \tag{10.1}$$

where

$k = permeability\ (m^2)$
$K = hydraulic\ conductivity\ (m/s)$
$\mu_d = dynamic\ viscosity\ (kg/ms)$
$\rho = density\ (kg/m^3)$
$g = gravitational\ acceleration\ (m/s^2)$

**Darcy's law**, given by Eq. 10.2, can be used to estimate the rate of flow of groundwater through a fully saturated porous rock matrix if the hydraulic conductivity and the hydraulic gradient are known.

$$Q = KA\frac{\Delta h}{\Delta l} \tag{10.2}$$

where

$Q$ = *flow* ($m^3/s$)
$K$ = *hydraulic conductivity* ($m/s$)
$A$ = *cross-sectional area* ($m^2$)
$\Delta h$ = *head difference along flow path* ($m$)
$\Delta l$ = *distance between head measurements* ($m$)

Fracture flow within strata units is more complex. For a wide range of fracture apertures, the so-called **cubic law** is often invoked. This law, defined by Eq. 10.3, is based on observed flow in smooth surface parallel plates and has been found to provide reasonable estimates of flow for fracture apertures as small as 4 µm (Witherspoon et al. 1980). However, rock fractures are rarely smooth. More often they exhibit rough surfaces (asperities), with variable apertures, tortuous flow paths and erratic connectivity to other fractures. Increasing fracture wall roughness and reducing apertures inevitably lead to a reduction in flow when compared to smooth parallel plate flow (Brown 1987).

$$Q = \frac{b^3 l \rho g}{12\mu_d} \frac{\Delta h}{\Delta l} \tag{10.3}$$

where

$b$ = *aperture* ($m$)
$l$ = *length of fracture orthogonal to flow* ($m$)

Regional scale analysis of flow within fracture networks is often based upon the determination of an equivalent porous medium (EPM). The analysis may be supported by numerical modelling based on fracture networks or stochastically generated fracture networks (Long et al. 1982).

In a layered system, such as a sedimentary rock mass, bedding and joint networks can result in conductivity within a stratum being transversely isotropic or anisotropic. In the absence of joints, the vertical conductivity of the system is controlled by rock formations with the lowest conductivity, while horizontal flow is controlled by rock formations with the highest conductivity. If a joint system pervades the bedded strata then the hydraulic conductivity of the system is altered and becomes a function of both the porous rock matrix and the joint attributes including aperture variability, asperities, spacing and network connectivity.

These attributes cannot be measured in situ but the impact on groundwater flows can be assessed and are predictable at both a local and regional scale if the rock matrix conductivities are known. In some coal basins, the impact of joints on groundwater flows is known to diminish with increasing confinement.

Hydraulic conductivity of strata is commonly assessed in situ by packer testing, or in the laboratory by core testing. Derived values are often incorporated in groundwater flow models in order to assess regional impacts of mining. Care is required when using packer testing to estimate hydraulic conductivity in vertical and sub-vertical boreholes as the technique is biased towards measuring the conductivity of intact rock and bedding plane features rather than vertical features. It reflects the local nature of the rock mass intersected by the borehole rather than the more regional nature of the rock mass in which the borehole is situated.

Piezometric monitoring is generally considered to be the best means for quantifying both the pre-mining hydrogeological regime and the impact of mining on this regime because it directly measures changes in the hydrogeological system. This type of monitoring has proven to be a sensitive indicator of change in conductivity and stress field well ahead of the mining face. Piezometric head falls with a reduction in compressive stress and with fracturing and extension of the rock mass and rises when the rock mass is subjected to an increase in compressive stress, when fractures close, and when new fractures fill with water over time. These responses and their significance are prone to misinterpretation.

In simple terms, inrush is usually associated with a directly connected and highly conductive conduit to a highly porous volume of storage, while groundwater inflow tends to be associated with seepage and drawdown over an extended period of time from a dispersed water source. Both sources of water inflow have the potential to impact on surface water systems and to result in inundation of mine workings. Fault zones should not be overlooked as possible conduits for inrush and inundation and should be monitored in the long term for change in conductivity associated with changes in mining-induced stress and erosion of flow paths.

### 10.3.2 Subsurface Effects

Underground coal mining is susceptible to subsidence-related inrush events (sudden inflow – discussed in more detail in Sect. 11.6.1) and inundation events (flooding over time). There are many examples in the literature, such as those described by Reynolds (1976), Xiao et al. (1991) and Byrnes (1999). There is considerable overlap between those subsidence effects that impact on the potential for inrush and inundation and those that impact on groundwater and surface water systems, with the main differences usually (but not always) relating to the scale and duration of impact. The management of water inflow from surface water bodies and aquifers to prevent inrush, inundation and adverse environmental impacts has traditionally been based on legislation and guidelines that involve one or a combination of the following controls:

- Minimum thickness of solid rock head (that is, unweathered rock). This thickness is primarily a function of mining method, mining height, strata composition and the presence and nature of geological structure and is typically in the range of 40–60 m. However, Byrnes (1999) has reported one case of a cover depth of 290 m (Appendix 11, Table A11.1 – solid rock head not specified).
- Minimum thickness of aquitard strata. Most often, this comprises clay rich units between the base of the water body and solid rock head but it can also include aquitards within the interburden.
- Maximum tensile strain at rock head, the typical range being 4–10 mm/m.
- Maximum mining height, which is often expressed as a fraction of the solid rock head cover thickness and indirectly linked to a limiting value of tensile strain at rock head.
- The development of a constrained zone within the superincumbent strata, characterised by minimal enhancement of vertical conductivity and enhanced horizontal conductivity over limited distances.

These controls tend to be empirical in nature, as evident in the summaries presented in Tables A11.1 and A11.2 of Appendix 11. They are neither mutually exclusive nor absolute. For example, the Reynolds Inquiry into mining beneath stored waters in NSW (Reynolds 1976) was presented with evidence of successful mining under Rend Lake, a large water reservoir in the USA. The lake bed was covered by many metres of sediment, comprised mainly of clay and silt and the interburden contained a high proportion of shale. Effectively, a 2 m thick seam was totally extracted at a depth of 200 m with no apparent adverse effect on the integrity of the water storage.

However, the Inquiry was also presented with the case of Rufford Lake in the UK, which was undermined at a depth of 420 m by longwall panels. The 200–250 m wide panels were contiguous and extracted to a height of 1.8 m. The overburden included a major sandstone aquifer overlain by near-surface limestone that was heavily fractured and fissured. About 4 years after the commencement of mining, the lake was drained through a mining-induced crack in the reservoir bed, although no serious problems were experienced with additional water entering the mine. Longwall mining continued, with new cracks and fissures of up to 200 mm in width appearing in the reservoir bed. The water loss was attributed to a suspected fault in the area and to concentrated tensile strains associated with a break in the continuity of the longwall panels.

The increased emphasis now placed on predicting ground subsidence effects and impacts and, in particular, on quantifying and managing impacts on surface water bodies, groundwater systems, and their dependent ecosystems, requires a basic understanding of subsurface ground behaviour mechanisms. The rapidly emerging importance of groundwater impacts and their consequences is reflected in the environmental assessment determinations of regulators such as DoP (2009, 2010).

A generalised conceptual model of rock mass behaviour above caved workings is shown in Fig. 3.5. The transition from a caved excavation

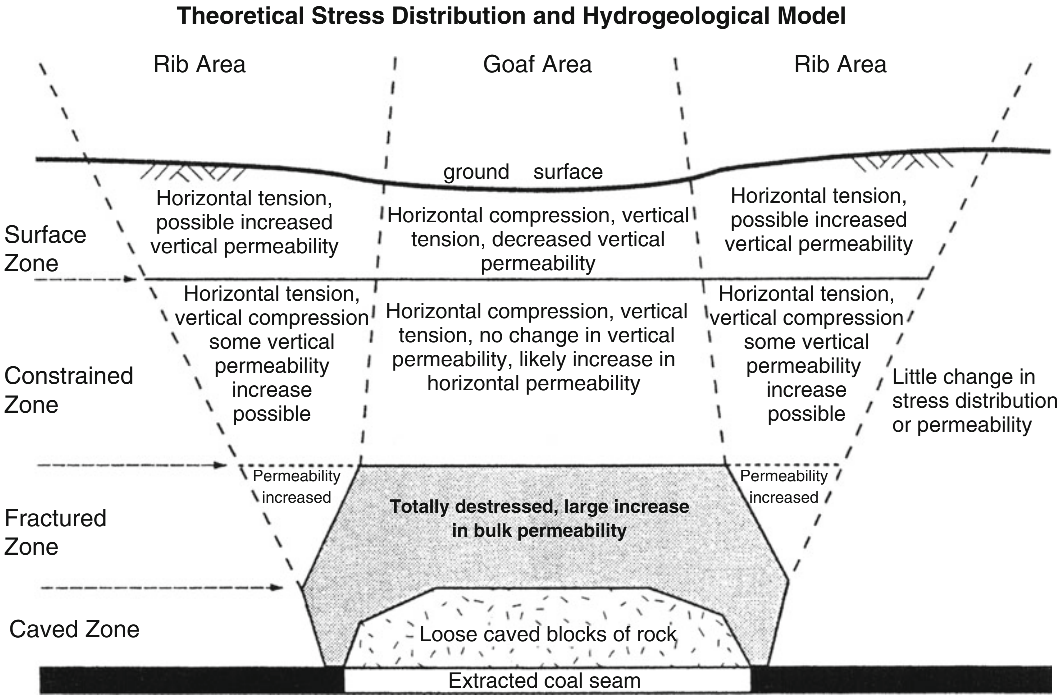

Theoretical Stress Distribution and Hydrogeological Model
Rib Area
Goaf Area
Rib Area
ground surface
Surface Zone
Horizontal tension, possible increased vertical permeability
Horizontal compression, vertical tension, decreased vertical permeability
Horizontal tension, possible increased vertical permeability
Constrained Zone
Horizontal tension, vertical compression some vertical permeability increase possible
Horizontal compression, vertical tension, no change in vertical permeability, likely increase in horizontal permeability
Horizontal tension, vertical compression some vertical permeability increase possible
Little change in stress distribution or permeability
Fractured Zone
Permeability increased
Totally destressed, large increase in bulk permeability
Permeability increased
Caved Zone
Loose caved blocks of rock
Extracted coal seam

**Fig. 10.3** Theoretical hydrogeological model proposed by Forster and Enever (1991) for mining in the Central Coast region of NSW

to a solid abutment results in a step dislocation, or shear plane, in the overburden close to the mining horizon. This disruption provides an enhanced conduit for water flow, with the degree of disruption decreasing with distance into the roof. Given sufficient depth, the upper roof strata ultimately behave as a series of plates subjected to curvature across this so-called **flexure zone**.

In utilising a four zone model to introduce caving mechanics in Sect. 3.3.1, it was noted that the model is not complete or universally accepted. Based on physical modelling, Whittaker and Reddish (1989) defined only two zones for hydrogeological purposes, being a zone of continuous cracking and a zone of discontinuous cracking. Forster and Enever (1991) developed a model for the Central Coast region of NSW based on four zones (Fig. 10.3). Thickness ranges assigned to these zones on the basis of field measurements over a supercritical mining layout are shown in Fig. 10.4. Although this hydrogeological model identified a potential for minor increase in vertical permeability over panel abutments, this increase was not detected in the testing program (Forster 1995). Examples of the values assigned by various researchers to the thickness of individual zones in a four zone model are presented in Table 11.2 of Appendix 11. The range in values reflects the influence on caving behaviour of both geology (particularly lithology) and variation in mining practice (such as size and number of remnant pillars in the goaf).

To address dichotomy and confusion between hydrologists and mining engineers, Kendorski (1993) proposed a five zone model based on dividing the constrained zone into a lower dilated zone extending up to 60 times extraction height, and an overlying constrained zone extending to within 17 m of the surface (Fig. 10.5). The dilated zone was postulated to be impacted by disconnected mining-induced fracturing, creating increased **storativity** (storage capacity) that initially results in a drop in water level. This recovers with time as the fractures fill and/or close when mining progresses away from the area. According to Kendorski (2006), the constrained zone is unaffected by subsidence

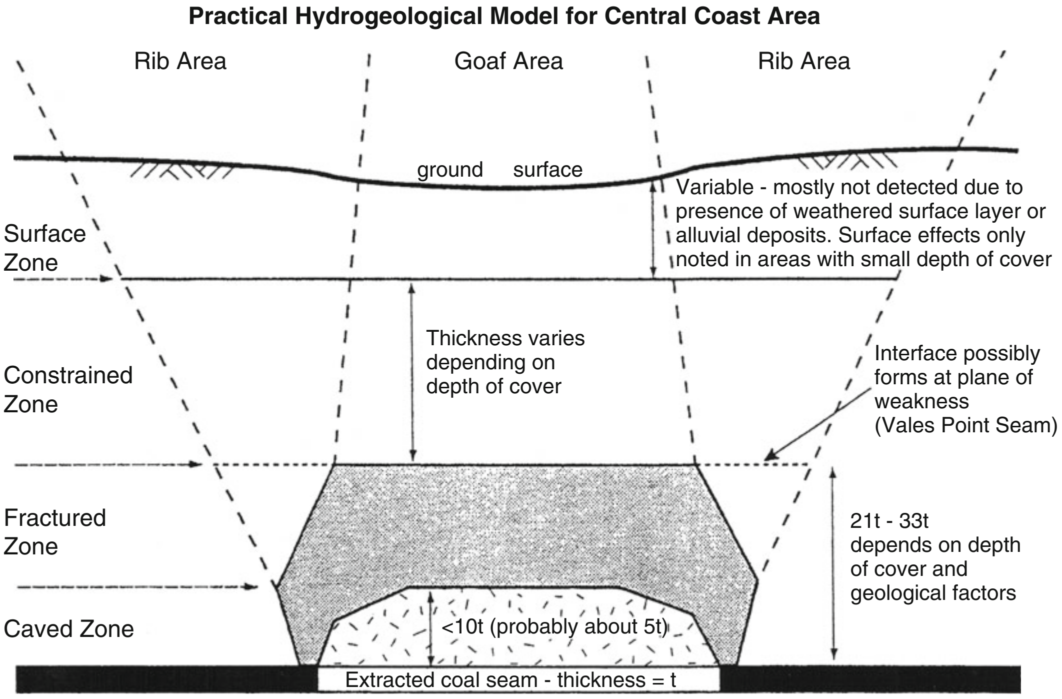

Practical Hydrogeological Model for Central Coast Area
Rib Area
Goaf Area
Rib Area
ground surface
Variable - mostly not detected due to presence of weathered surface layer or alluvial deposits. Surface effects only noted in areas with small depth of cover
Surface Zone
Constrained Zone
Thickness varies depending on depth of cover
Interface possibly forms at plane of weakness (Vales Point Seam)
Fractured Zone
21t - 33t depends on depth of cover and geological factors
Caved Zone
<10t (probably about 5t)
Extracted coal seam - thickness = t

**Fig. 10.4** Zone thickness ranges proposed on the basis of field monitoring for the NSW Central Coast theoretical hydrogeological model (After Forster and Enever 1991)

deformation and experiences no change in permeability.

The constrained zone is sometimes considered and referred to as an 'aquiclude zone'. This is a matter of definition and variance between the different professions involved in subsurface behaviour. Even if the pre-mining permeability prevails in this zone, from a hydrogeological perspective this permeability may still not qualify it to be considered as an aquiclude. Table 11.3 of Appendix 11 lists a range of recommendations related to the presence of low permeability strata within a constrained zone.

Quantifying caving behaviour has been aided by a number of studies that utilised multi-point surface-to-seam wire extensometers, notably Hardman (1968), Gurtunca (1984), Holla and Armstrong (1986), Mills and O'Grady (1998) and Shen et al. (2014). When installing these instruments, care is required to avoid interference between the wires as this can result in spurious displacement readings. The interpretation of measurements also has to recognise that shear movement above the caved zone and block rotation within the caved zone and fractured zone can comprise an unknown portion of the measured displacements. This can sometimes result in 'measured' vertical displacement exceeding the extracted seam thickness.

Mills and O'Grady (1998) utilised an array of four, ten point extensometers across two longwall panels of differing subcritical width to deduce the subsurface vertical displacement distributions shown in Fig. 10.6. It was concluded that the zone of large vertical movement extended to a height of approximately 1.0–1.1 times the panel width, with movement occurring in discrete blocks. Separation of the blocks was concentrated at distances of 20, 50, 100 and 130 m above the mining horizon. A range of other surface to seam extensometer monitoring (for example, Hardman 1968, and Holla 1989) has also identified zones of large strata dilation intermixed with zones of low strata dilation (or en masse or block movement), confirming that caving and subsidence is not always a continuous process.

Further insight into subsurface behaviour is provided by the research of Guo et al. (2007)

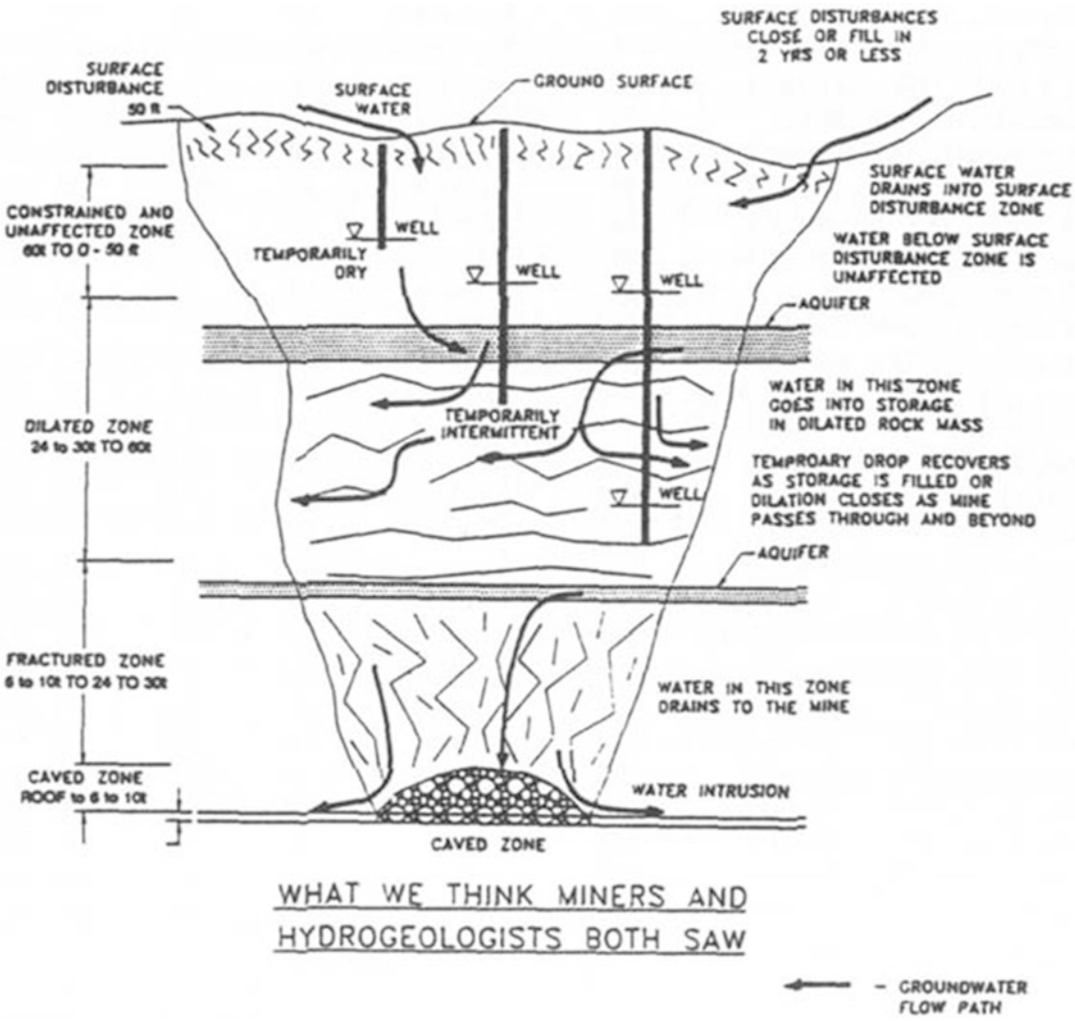

SURFACE DISTURBANCES CLOSE OR FILL IN 2 YRS OR LESS
SURFACE DISTURBANCE 50 ft
SURFACE WATER
GROUND SURFACE
SURFACE WATER DRAINS INTO SURFACE DISTURBANCE ZONE
CONSTRAINED AND UNAFFECTED ZONE 60t TO 0 - 50 ft
WELL
TEMPORARILY DRY
WELL
WELL
WATER BELOW SURFACE DISTURBANCE ZONE IS UNAFFECTED
AQUIFER
WATER IN THIS ZONE GOES INTO STORAGE IN DILATED ROCK MASS
DILATED ZONE 24 to 30t TO 60t
TEMPORARILY INTERMITTENT
TEMPROARY DROP RECOVERS AS STORAGE IS FILLED OR DILATION CLOSES AS MINE PASSES THROUGH AND BEYOND
WELL
AQUIFER
FRACTURED ZONE 6 to 10t TO 24 TO 30t
WATER IN THIS ZONE DRAINS TO THE MINE
CAVED ZONE ROOF to 6 to 10t
WATER INTRUSION
CAVED ZONE
WHAT WE THINK MINERS AND HYDROGEOLOGISTS BOTH SAW
- GROUNDWATER FLOW PATH

**Fig. 10.5** A conceptual five zone model of caving and fracturing above an excavation (After Kendorski 1993)

The researchers integrated geological information, a range of piezometer, extensometer and other field monitoring data, and a three-dimensional coupled numerical model to develop a hydrogeological model for Springvale Colliery, Australia. The piezometric monitoring indicated that the zone of influence of longwall mining above a 260 m wide panel and a 315 m wide panel extended to 250 and 275 m, respectively, above the roof of the extraction horizon. The influence of mining was detected up to 350 m in advance of the face. No effect on piezometric pressure over the 315 m wide panel was detected at depths of 30 and 50 m below surface, corresponding to 344 and 365.6 m above the roof of the extraction horizon. The research led to the postulation of a six zone sub-surface model that included an **elastic zone** overlying the constrained zone (Fig. 10.7). The calibrated numerical model predicted that water inflow rates for the specific hydrogeological conditions would increase by 84 % if panel width was increased from 240 to 320 m, and by 41 % if mining height was increased from 3 to 4.5 m.

The variation in conductivity above an extraction panel for a range of depths and panel geometries was examined by Gale (2008) utilising numerical modelling of overburden caving and subsidence behaviour, validated by

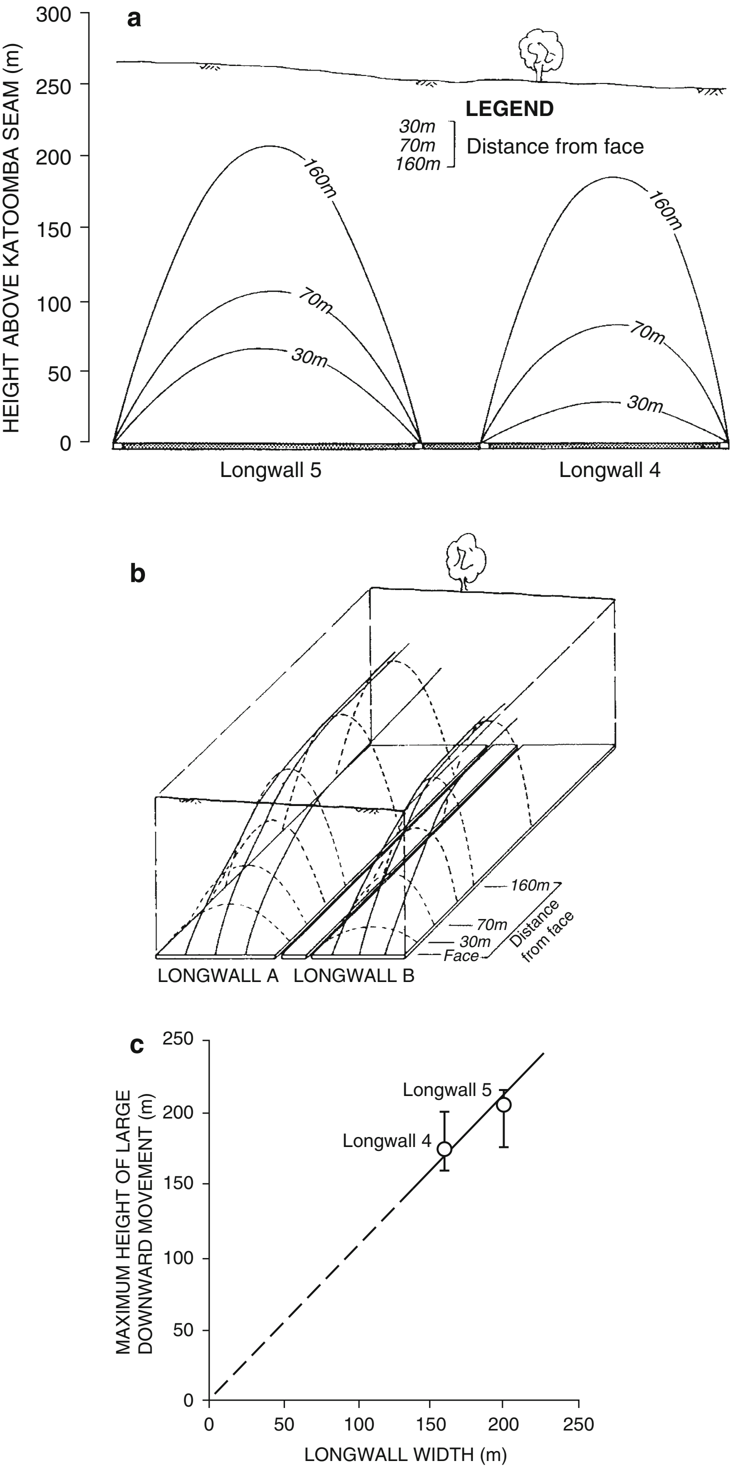


**Fig. 10.6** Zones of large downward displacement measured above two longwalls of different subcritical width. (**a**) Cross-sections, (**b**) Schematic isometric of completed longwalls, (**c**) Relationship between longwall panel width and height of the zone of large downward movement (After Mills and O'Grady 1998)

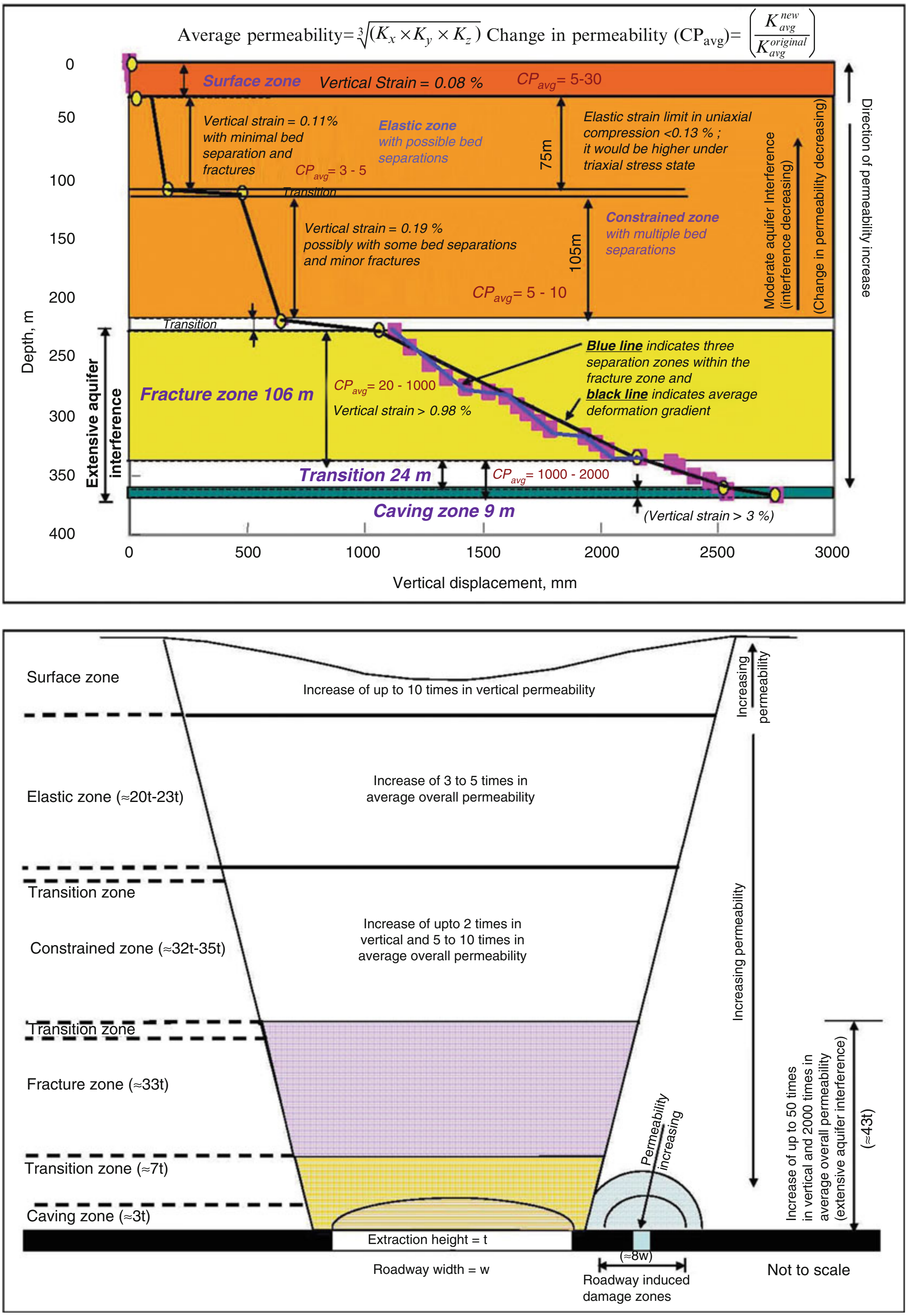

Average permeability = $\sqrt[3]{(K_x \times K_y \times K_z)}$ Change in permeability ($CP_{avg}$) = $\left(\frac{K_{avg}^{new}}{K_{avg}^{original}}\right)$
Surface zone
Vertical Strain = 0.08 %
$CP_{avg}$ = 5-30
Vertical strain = 0.11% with minimal bed separation and fractures
$CP_{avg}$ = 3 - 5
Elastic zone with possible bed separations
75m
Elastic strain limit in uniaxial compression <0.13 % ; it would be higher under triaxial stress state
Transition
Vertical strain = 0.19 % possibly with some bed separations and minor fractures
105m
Constrained zone with multiple bed separations
$CP_{avg}$ = 5 - 10
Moderate aquifer Interference (interference decreasing)
(Change in permeability decreasing)
Direction of permeability increase
Transition
Fracture zone 106 m
$CP_{avg}$ = 20 - 1000
Vertical strain > 0.98 %
Blue line indicates three separation zones within the fracture zone and black line indicates average deformation gradient
Transition 24 m
$CP_{avg}$ = 1000 - 2000
Caving zone 9 m
(Vertical strain > 3 %)
Extensive aquifer interference
Depth, m
0
50
100
150
200
250
300
350
400
0
500
1000
1500
2000
2500
3000
Vertical displacement, mm
Surface zone
Increase of up to 10 times in vertical permeability
Increasing permeability
Elastic zone (≈20t-23t)
Increase of 3 to 5 times in average overall permeability
Transition zone
Constrained zone (≈32t-35t)
Increase of upto 2 times in vertical and 5 to 10 times in average overall permeability
Increasing permeability
Transition zone
Fracture zone (≈33t)
Increase of up to 50 times in vertical and 2000 times in average overall permeability (extensive aquifer interference)
(≈43t)
Permeability increasing
Transition zone (≈7t)
Caving zone (≈3t)
Extraction height = t
Roadway width = w
(≈8w)
Roadway induced damage zones
Not to scale

**Fig. 10.7** Hydrogeological model developed for Springvale Colliery, Australia, by Guo et al. (2007)

back-analysis of site data. Modelled average conductivity was found to correlate well with the measured values, with Gale concluding that the modelling process had simulated the fracture and flow systems. It was concluded that caving and cracked beam subsidence movements tend to occur up to a height of 1–1.7 times the panel width (Fig. 10.8). Gale noted that examples of this behaviour had been monitored by surface to seam extensometers (Holla and Armstrong 1986; Mills and O'Grady 1998; Hatherly et al. 2003; Guo et al. 2005) and were consistent with earlier numerical modelling by Gale (2006). It is not inconsistent with microseismic monitoring undertaken by Kelly and Gale (1999) and which recorded no events more than 290 m above 200 m wide longwall panels in the Southern Coalfield of NSW, Australia (see Fig. 3.18).

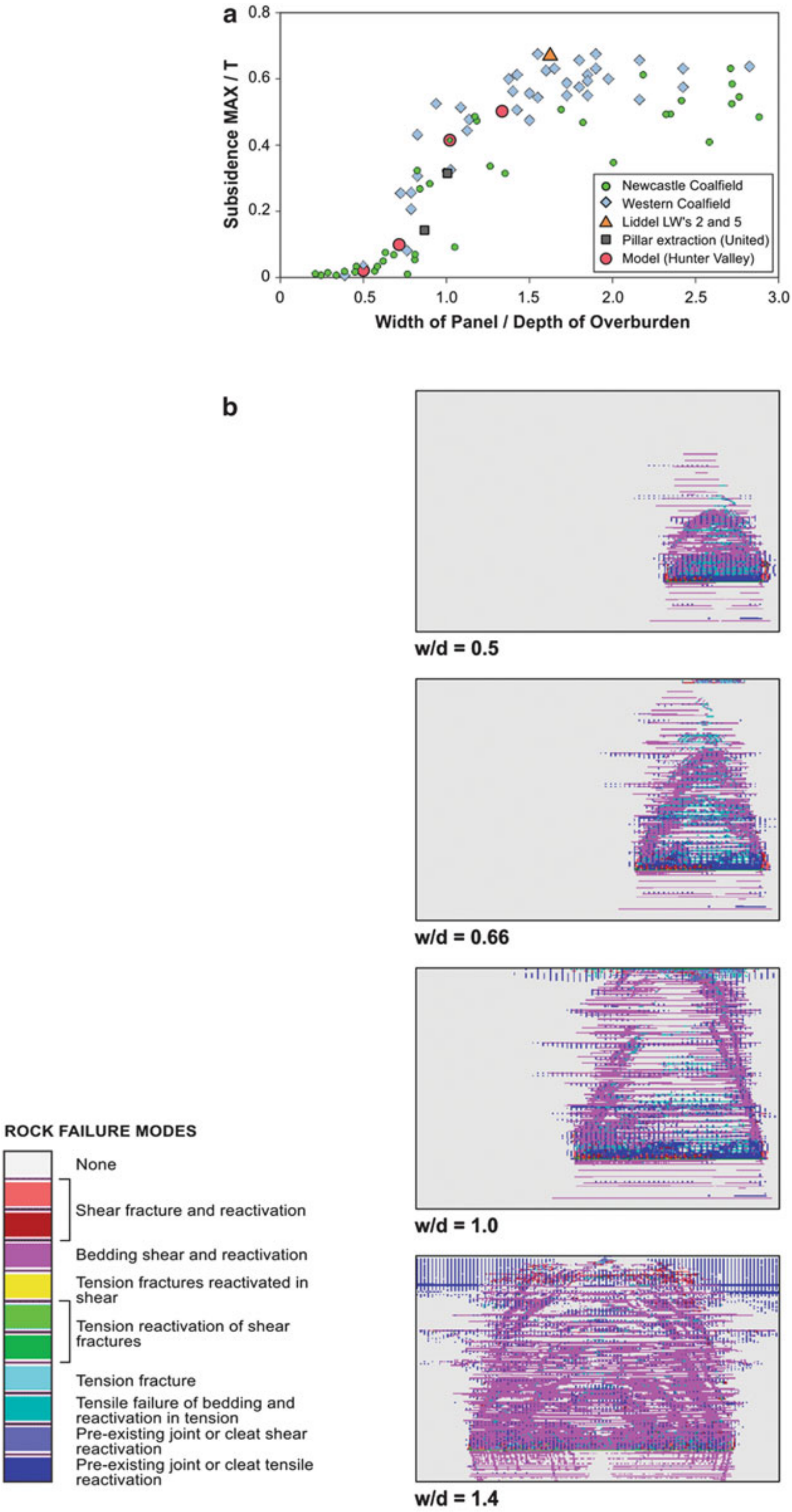
The numerical modelling by Gale (2008) formed the basis for utilising field data to develop the relationship between conductivity and surface strain shown in Fig. 10.9. The effect of mining height and panel width are embedded in the measured vertical surface displacement. The modelling results indicated that for the case of 4 mm/m tensile strain at the surface, the overburden had flow networks close to the in situ conductivity and, therefore, this strain value provided a reasonable estimate for the onset of enhanced conductivity of the overburden. It was noted that an hydraulic profile was maintained in the upper strata when the average conductivity was less than approximately $10^{-6}$ m/s. This was believed to indicate that flow in strata with an average conductivity of less than $10^{-6}$ m/s is tortuous and may support a water table under the appropriate conditions.

An hydraulic profile was not maintained when the average conductivity was greater than approximately $10^{-5}$ m/s, with the overburden predicted to support only localised perched water above the goaf. In the case where surface strain above a panel was 10 mm/m, predicted average conductivity of overburden was typically in the range of $10^{-2}$–$10^{-3}$ m/s. Gale's modelling showed that the conductivity of the overburden was not impacted further than 50 m from the ribline and that significant increase in conductivity of the overburden due to mining could be expected to be limited to a zone less than the angle of draw (which defines the lateral extent of mining-induced vertical displacement on the surface – see Sect. 10.4.3). However, the pore pressure distribution may be modified.

It was concluded that fracture connectivity was greater in stiff, sandstone-rich strata relative to strata having many coal and tuffaceous units. This was related to the ability of the less stiff overburden to flex and displace onto the goaf rather than to fracture and rotate about the ribsides. Gale (2008) also concluded from the modelling that while panel width typically controlled the height of fracturing, the network connectivity and conductivity of fractures was controlled by the magnitude of strain and vertical surface displacement as determined by panel width, panel depth and seam thickness. Figure 10.10 shows changes in vertical conductivity with depth predicted from the numerical modelling for three different magnitudes of maximum vertical surface displacement.

The research findings of Gale (2008) were premised on a number of important qualifications. Firstly, the empirical strain values input to the numerical modelling did not relate to actual measured strain but rather to the equivalent theoretical uniformly distributed strain. That is, it was assumed that strain would be uniformly distributed over the stated measurement interval (1 m) rather than, as often happens in practice, concentrated at specific sites over several strain measurement intervals. Secondly, although the findings have been presented with a focus on water inflows sufficient to interfere with mining operations, they find direct application to assessing groundwater impacts. Thirdly, the data upon which the findings are premised typically relate to single panels or multiple panels at shallow depth where chain pillar compression is not significant. In deeper mines where individual panels are of subcritical width and chain pillar and surrounding strata compression or yield is significant, numerical modelling of the overall mining geometry is recommended. The research did not consider

**Fig. 10.8** Strata fracture for various total extraction panel geometries as determined by Gale (2008) from numerical modelling, and comparison of measured vertical surface displacement with modelling predictions. (**a**) General relationship, (**b**) Modelled strata fracture ('T' = mining height, 'w' = panel width, 'd' = depth of mining, 'Subsidence' = vertical surface displacement)



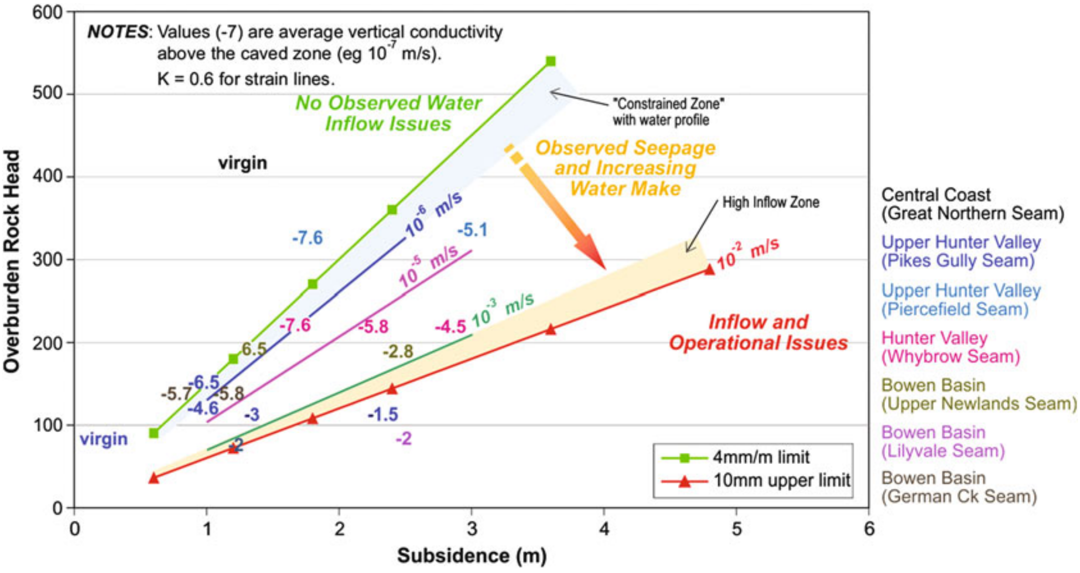


**Fig. 10.9** Average overburden conductivity characteristics relative to vertical displacement (subsidence) and depth criteria proposed by Gale (2008) – ('Subsidence' = vertical surface displacement)

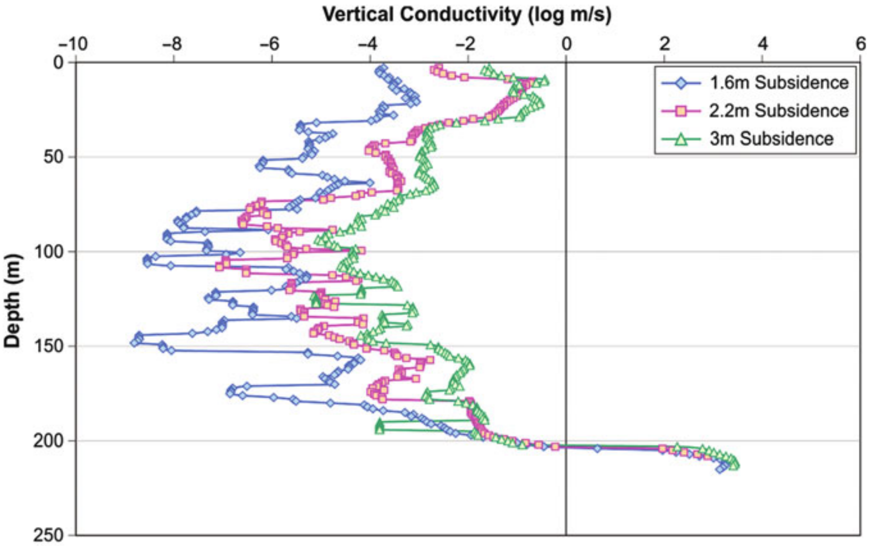


**Fig. 10.10** Conductivity changes related to increased maximum vertical surface displacement (subsidence) for a constant panel width as predicted by Gale (2008) utilising numerical modelling ('Subsidence' = vertical surface displacement)

the nature of the rock matrix. In some situations, high rates of vertical drainage could be initiated without strain induced cracking of the strata.

Many of the research findings noted to this point are encapsulated in the six zone sub-surface subsidence model proposed by Mills (2012) on the basis of subsidence monitoring, borehole camera observations, packer testing, piezometer data, microseismic data, extensometer monitoring, and stress change monitoring. This model is shown in Fig. 10.11. It depicts zones of fracturing and might be considered to provide the

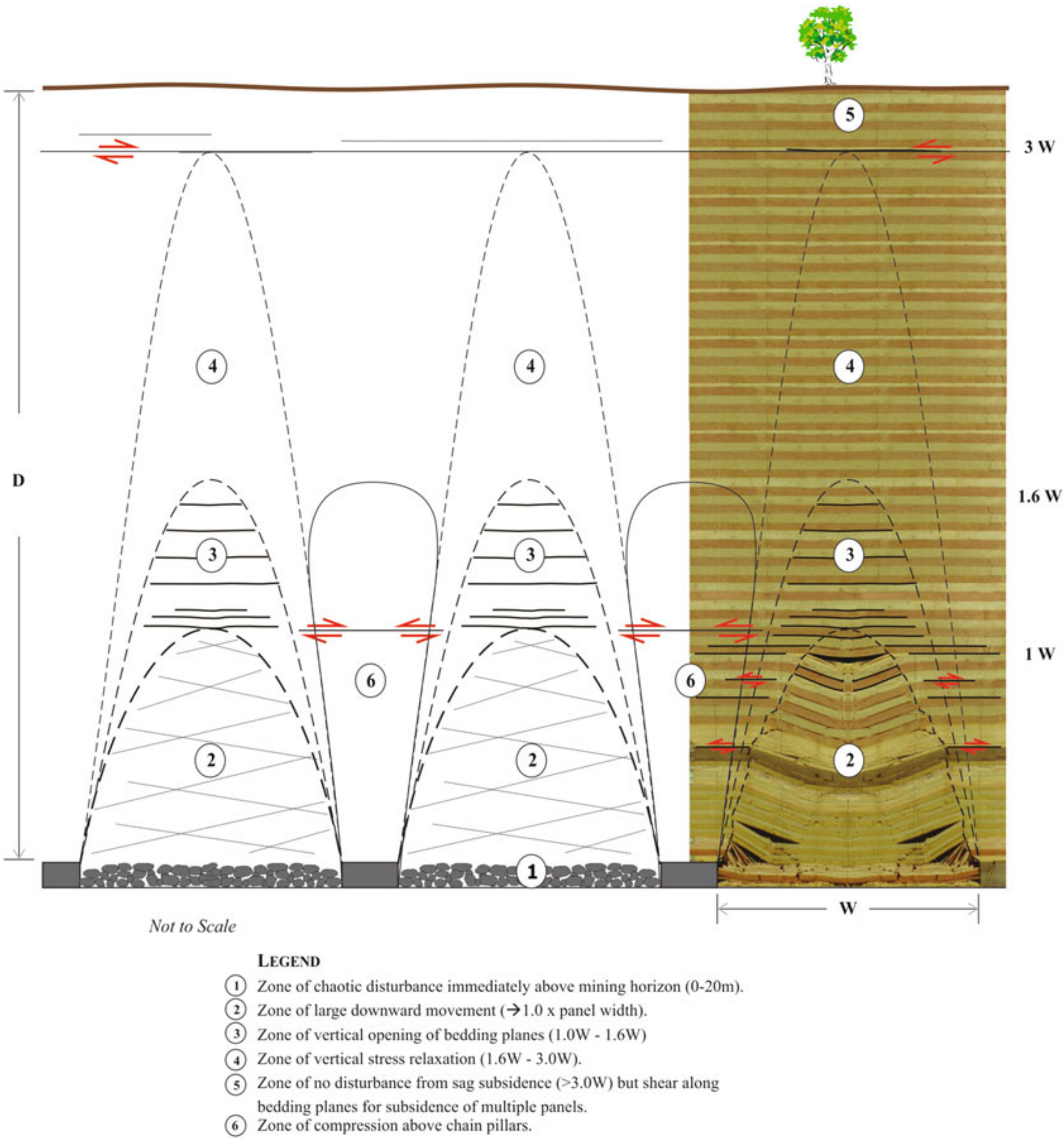

5
3 W
4
4
4
D
1.6 W
3
3
3
1 W
6
6
2
2
2
1
W
Not to Scale
LEGEND
① Zone of chaotic disturbance immediately above mining horizon (0-20m).
② Zone of large downward movement (→1.0 x panel width).
③ Zone of vertical opening of bedding planes (1.0W - 1.6W)
④ Zone of vertical stress relaxation (1.6W - 3.0W).
⑤ Zone of no disturbance from sag subsidence (>3.0W) but shear along bedding planes for subsidence of multiple panels.
⑥ Zone of compression above chain pillars.

**Fig. 10.11** Overburden caving model inferred by Mills (2012) on the basis of accumulated surface and subsurface monitoring data from a range of sources

most generally applicable conceptualisation of subsurface ground movements at typical coal mining depths. Six zones have been characterised on the basis of the nature and level of strata disturbance, these being:

- Zone 1 – a zone of chaotic disturbance immediately above the mining horizon, corresponding to the caving horizon.
- Zone 2 – a zone extending from the seam horizon to a height approximately equal to the panel width and characterised by extensive conjugate shear fracturing with numerous open fractures, particularly around the margins of the zone, and numerous inclined fractures throughout.
- Zone 3 – a transitional zone occupying the vertical interval from approximately 1.0

times panel width to 1.6 times panel width above the mining horizon and comprising moderate ground fracturing and dilation characterised by the vertical opening of horizontal bedding planes with horizontal fractures being dominant in fracture logs. This is described as typically a zone of slight depressurisation consistent with low level flow in a downward direction, with recharge from rainfall sometimes sufficient to substantially maintain groundwater levels in the upper levels of the overburden.
- Zone 4 – the interval from approximately 1.6 times panel width to 3.0 times panel width above the mining horizon, in which vertical displacements are consistent in magnitude with elastic relaxation of the pre-mining vertical stresses without the need for physical opening of bedding planes.
- Zone 5 – a zone above about three times panel width above the mining horizon that is characterised by no ground fracturing and dilation but shear along bedding planes. Mills (2012) noted, however, that in a multiple longwall panel situation, elastic strata compression typically makes a significant contribution to subsidence, resulting in differential shearing on bedding planes within this upper zone that contributes to stress relief movements.
- Zone 6 – a zone of compression above interpanel pillars.

The zones above each panel were considered to be arch-shaped on the basis of the extensometer monitoring conducted by Mills and O'Grady (1998). The height of Zone 2 was projected to increase with increase in horizontal stress. Elastic compression of the abutments and strata surrounding the interpanel panels has been credited with reducing fracture volume and hydraulic conductivity in these regions, especially in the vertical direction. The interface between the side of Zone 6 and Zones 1 and 2 was noted to accommodate relatively large differential movements of rock strata over a short distance and was said to be characterised by open shear fractures and fractures between rotated blocks of intact material.

Tammetta (2013) proposed an empirical equation (Eq. 10.4) for calculating the height of complete groundwater drainage, $H_d$, that attempts to take account of excavation width, W, mining height, h, and depth of mining, H. Equation 10.4 was derived by regression analysis of ground movement detected by multi-point borehole extensometers at 18 mines from nine coalfields in five countries. It is stated to have a 5 % probability bound at 31 m below the predicted height of complete groundwater drainage and a 95 % bound at 37 m above the predicted height. The relationship is not accepted universally, with a number of aspects warranting further research. These include the reliability of determining the horizon of zero pressure on the basis of extensometer data; the capacity to distinguish between depressurisation and zero flow; and how to also account for the effect of time on groundwater drainage height.

$$H_d = 1438\, ln\left[4.315 \times 10^{-5}\left(Wh^{1.4}H^{0.2}\right) + 0.9818\right] + 26 \tag{10.4}$$

where

$H_d = height\ of\ complete\ groundwater\ drainage$

Empirically deduced models of sub-surface behaviour zones are very useful for conceptualising the development of subsidence effects and their impacts and, in many cases, may prove to be quite accurate predictors. However, the end-user needs to be alert to a range of limitations associated with these models, the more important being:

- None have regard to the effect of horizontal-to-vertical stress ratio on strata behaviour, even though, as evident in preceding chapters, this parameter is important when considering excavation performance. It is an important consideration because it can impact on permeability, conductivity and the formation of a constrained zone.

- None have regard to discontinuous subsidence associated with bridging strata. Should a constrained zone develop as a result of caving and fracturing being interrupted by the presence of a particularly competent bed that spans the excavation, extrapolation of the corresponding derived zone thicknesses to different geological settings is fraught with risk. This is complicated by the fact that the capacity for bridging is also a function of the horizontal-to-vertical stress ratio which, as noted already, is not taken into account by the models.
- Although it is convenient to divide sub-surface behaviour into a series of zones with distinct physical and/or hydrogeological characteristics, in reality behaviour types, permeability and the lateral extent of affected areas change gradationally as depth of mining increases relative to panel width. Furthermore, due to site-specific properties, this transformation can vary within a broad spectrum of panel width-to-depth ratios, as evidenced by the zone boundaries suggested by Mills (2012), and over extensive thickness ranges, as evidenced in most subsurface behaviour models.

Each of the sub-surface fracture models reviewed in this Chapter has limitations. For example, the four zone model shown in Fig. 10.4 does not factor in panel width when calculating the height of the constrained zone above the mining horizon. This is of no concern when applied to conditions similar to those used to derive the model. However, in different conditions, determining the distance to the base of the constrained zone only on the basis that it 21–33 times the extraction height may prove unreliable. Conversely, experience associated with mining deep tabular deposits confirms engineering logic that the distance of the constrained zone above the mining horizon will not continue to increase indefinitely with increasing panel width in accordance with the six zone model depicted in Fig. 10.11. This model appears to work well for typical extraction panel widths currently utilised in some underground coal mining districts in Australia, but cannot be presumed to apply to wider panels.

Irrespective of which approach or model is adopted to characterise the conductivity of the superincumbent strata, careful consideration must always be given to the potential for geological features to enhance water inflow. These features can range in scale from natural joint systems through to dykes and faults and while all may result in inflow, an elevated risk is usually associated with the latter. By way of example, Byrnes (1999) reported that a dyke was the possible point of leakage from the Avon Reservoir into Wongawilli Mine, Australia, even though the workings were outside a 35° angle of draw and tensile and compressive strains were only 1.41 and 1.65 mm/m, respectively. Mining at a depth of 420 m in another case was also noted to cause fracturing in the bed of an overlying reservoir, with significant loss of water. The cracking was suspected to be due to the reopening of a fault plane beneath the lake.

On the other hand, as previously noted, Anderson et al. (1989) reported a case of extensive longwall mining at a depth of 200 m beneath a freshwater lake without any reported loss of water, despite the presence of a number of fault zones that were continuous between the surface and the mine workings. Significantly, the floor of the lake contained many metres of clay and silt. Similarly, Van Roosendaal et al. (1995) attributed shale and clay overburden with preventing the loss of aquifer water above longwall panels at a depth of only 60 m in Illinois, USA. Seedsman (2006) and Gale (2008) reported similar experiences in regard to mining in the vicinity of Crinum Mine and Kestrel Mine in Queensland, Australia, where a clay-rich layer at the base of water-bearing basalt units is reported to have acted as an aquiclude between the aquifer and the underlying fractured strata above longwall goaves. When the interburden thickness to the aquifer was reduced to 70 m, an inrush occurred (Payne 2008). The capacity for clay to impede water inflow is widely attributed in the literature to its low permeability, its capacity to strain without fracturing and its potential to self-heal.

### 10.3.3 Impacts

Sub-surface behaviour has the potential to impact on:

- inrush into the mine workings of fluids and materials that flows when wet;
- inundation (flooding) of the mine workings and, hence, pumping requirements and mine stability in the long term;
- the stability of existing and future overlying workings;
- groundwater and surface water flows and quality;
- environmental, aesthetic and amenity attributes associated with surface and groundwater systems.

The risk of inrush from a source within the superincumbent strata is related usually to either:

- the first instance of caving extending up to an aquifer or fluid body, as in the case of the incident shown in Fig. 10.12;
- mining into a well developed conduit to a fluid source;
- relaxation (unclamping) of a geological feature that connects to a fluid source; or
- mining close to flooded workings.

The risk of inrush can take time to materialise since it may be related to the extent of mining or to time dependent behaviour, such as the development of piping. There can be a fine line between some of these events constituting an inundation event rather than an inrush. Based on UK experience, Wardell (1975) recommended that, as a control against inundation when working beneath tidal waters, no total extraction or pillar extraction should be permitted within a distance of 40 m of any known fault having a vertical displacement greater than 3 m nor any dyke having a width greater than 6 m. This recommendation appears to have worked well over a number of decades of mining beneath Lake Macquarie in Australia.

Inrush and inundation events may result in rapid flooding of a mine. Flooding can also

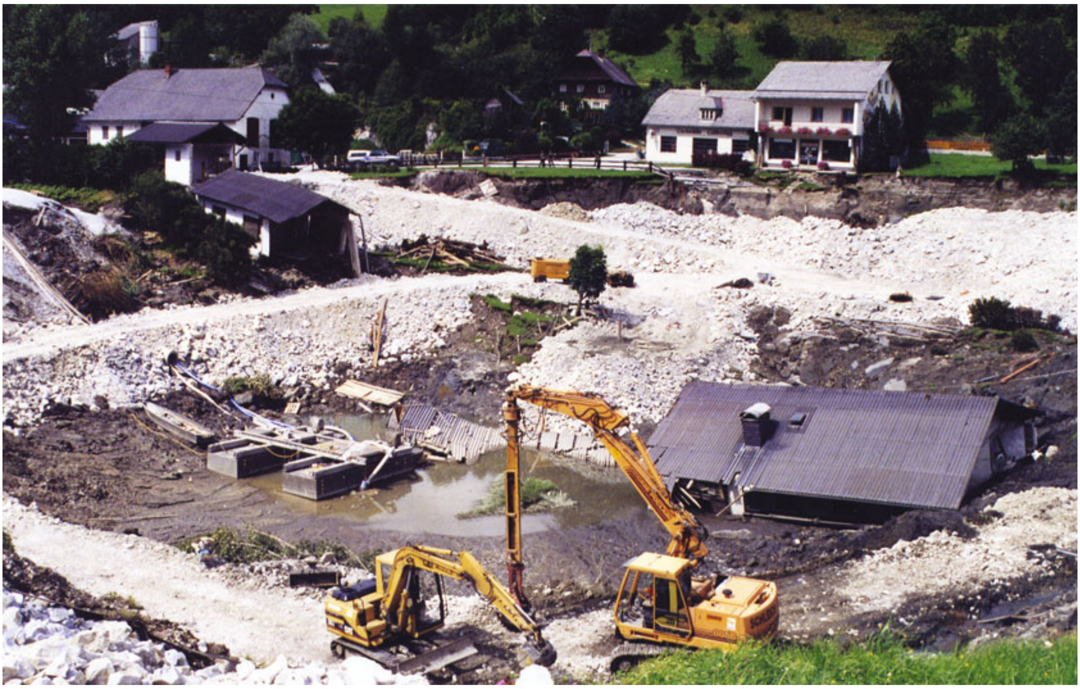


**Fig. 10.12** An example of caving above mine workings (talc) intercepting saturated alluvium that then flowed into mine – the white material is silica rock used to stabilise inflow (After Galvin 1998)

develop over time due to groundwater make from the exposed perimeter of the coal seam and the strata in the caved and fractured zones; seepage from strata in the constrained zone; and direct and indirect hydraulic conduits to surface water bodies.

Sub-surface subsidence can have impacts on future workings in overlying seams by fracturing the roof, coal and floor strata of these workings above extraction panels in the lower seam; generating elevated stress concentrations above interpanel pillars and solid abutments in the lower seam; and creating flexure zones above lower seam abutments. The extent of fracturing may be more severe in pre-existing upper seam workings because they are in an unconfined state when subsided due to mining in deeper seams.

Experience and numerical modelling have both shown that there is an elevated risk of strata instability and water inflow when mining through flexure zones in an upper seam. Stability is usually impacted more adversely when mining progresses from over an excavation in an underlying excavation to over the abutment of the lower seam workings. As illustrated in Fig. 5.20 this is associated with creating a progressively smaller window ahead of the working face through which lower seam concentrations can pass.

At shallow depths, it is possible for a flexure zone to be in a permanent state of extension, so that it provides the opportunity for direct hydraulic connection between the working horizon and overlying water bodies. As depth increases, the degree of disruption in the upper portion of the flexure zone decreases and the strata behave more as a series of plates subjected to bending. Depending on their location within the flexure zone, either the top or the bottom of these strata units will be subjected to tensile strain, leading to potential for tensile fracturing if the tensile bending stresses exceed the primitive compressive stresses. Although, theoretically, tension on one face of a strata unit will be balanced by compression on the other, bedding plane shear provides an enhanced capacity for indirect hydraulic connection between sets of tension cracks at shallow to moderate depth.

There is a history of flexure zones providing conduits for surface water to enter mine workings at shallower depths, typically less than 200 m. The risk is elevated in times of surface flooding due to the increased catchment area of flexure zones, the increased water head, and the 'unlimited' capacity of the water source. Subsidence management plans needs to recognise this and make provision for appropriate mitigation measures, inundation controls and contingencies.

Many of the preceding considerations are relevant to managing environmental impacts arising from subsurface subsidence. These impacts can vary across a wide range and may be associated with:

- loss of stream base flow due to drainage of the feeder groundwater system;
- diversion of surface water flows into subsurface fracture networks, mine workings or other catchments;
- loss of biodiversity due to changes in flow volumes, flow patterns and water quality;
- depressurisation of aquifers and associated reduction or loss of bore water;
- cracking and cross connection of aquifers;
- the generation of regional groundwater sinks.

The nature and extent of these impacts are influenced significantly by the composition, thickness and hydraulic properties of the lithology overlying mine workings and the presence or absence of a constrained zone. The earlier noted research by Gale (2008) concluded that a constrained zone formed within the overburden when the average conductivity was less than approximately $10^{-6}$ m/s. In some instances when the overburden conductivity has been greater than approximately $10^{-3}$ m/s, significant impacts on natural streams and aquifers have been experienced. Irrespective of the presence of a constrained zone, a mine void constitutes a sink. The rate and extent of drawdown of groundwater into it is controlled by the pre-mining hydraulic conductivity of the impacted strata and by the pressure gradient created by depressurising the strata at the working horizon in the mine.

Sub-surface fracturing can have implications for gas release from mine workings to the surface. Although conductivity and connectivity of fracture networks is greatest at shallow depth, release of coal seam gas through these networks does not appear to have been an issue at shallow depth. This may be because shallow seams have already drained over geological time. However, gas release has been associated with deep seams in the Southern Coalfield of Australia, resulting in bubbles surfacing in watercourses and dieback of vegetation in the immediate vicinity of gas vents. Signature testing of the gas identified that it originated from sandstone formations in the upper stratigraphic column and not the mining horizon (DoP 2010). However, it is possible that deep seam gas may have migrated vertically over geological time and accumulated in porous storage (Mackie 2014). There is a history of the gas releases in the Southern Coalfield of Australia dissipating over time and gas vent areas being satisfactorily revegetated (DoP 2008, 2010).

In summary, mining-induced changes in conductivity of the superincumbent strata are primarily a function of panel width-to-depth ratio and extraction height. It follows, therefore, that these two design parameters are primary controls for mitigating the impacts of sub-surface subsidence. Further research is required to better quantify these relationships. In the interim, it is advisable to adopt a conservative approach when applying existing relationships.

## 10.4 Surface Subsidence

### 10.4.1 Introduction

The behaviour of the ground close to and at the surface is not a simple and direct extension of that associated with sub-surface behaviour. This is because the ground surface is unconfined in the vertical direction and, in the case of steep topography, also laterally to some extent. The lack of confinement results in additional disturbance down to observed depths of typically 20 m, but sometimes greater. When mining occurs at a depth greater that about 80 m, the profile of vertical surface displacement nearly always takes the form of a trough. At shallower depths, particularly less than 30 m, it may also take the form of one or more steep sided, ovoid shaped depressions.

### 10.4.2 Sinkhole and Plug Subsidence

When a subsidence depression is stepped and confined to a small area, such as over a roadway intersection, it is referred to variously as a **sinkhole**, **chimney cave**, or **pothole**, examples of which are shown in Figs. 3.34, 3.35, 10.13a, b. A more regional step depression, typically over part or all of an extraction panel as shown Figs. 3.36 and 10.13c, d is generally referred to in underground coal mining as a **plug failure**. (Note the risk associated with the position of some personnel in these photographs may not be considered tolerable under contemporary risk management standards.)

Usually, there are no degrees of tolerance regarding the occurrence of sinkholes and plug failures. Either the step change in surface profile and its associated impacts can be tolerated in the given circumstances or they cannot. Hence, risk assessment tends to focus on identifying the potential for these types of failures rather than predicting the magnitude and distribution of their impacts.

Most sinkholes develop by the progressive collapse of the roof strata, or formation of a chimney, to the surface. Factors that influence the likelihood and extent of sinkholes include mining height, void space in the workings, bulking factor, groundwater, surface water, and the composition of the overburden. Thinly laminated strata are particularly prone to sinkhole formation. The height of the chimney required for the caving process to progress to the point where it becomes choked off and is self-arrested increases with increase in mining height and with decrease in bulking factor. Caving can also be arrested by a strong bed in the overburden. The risk of sinkhole development is increased in the presence of water, as reflected in

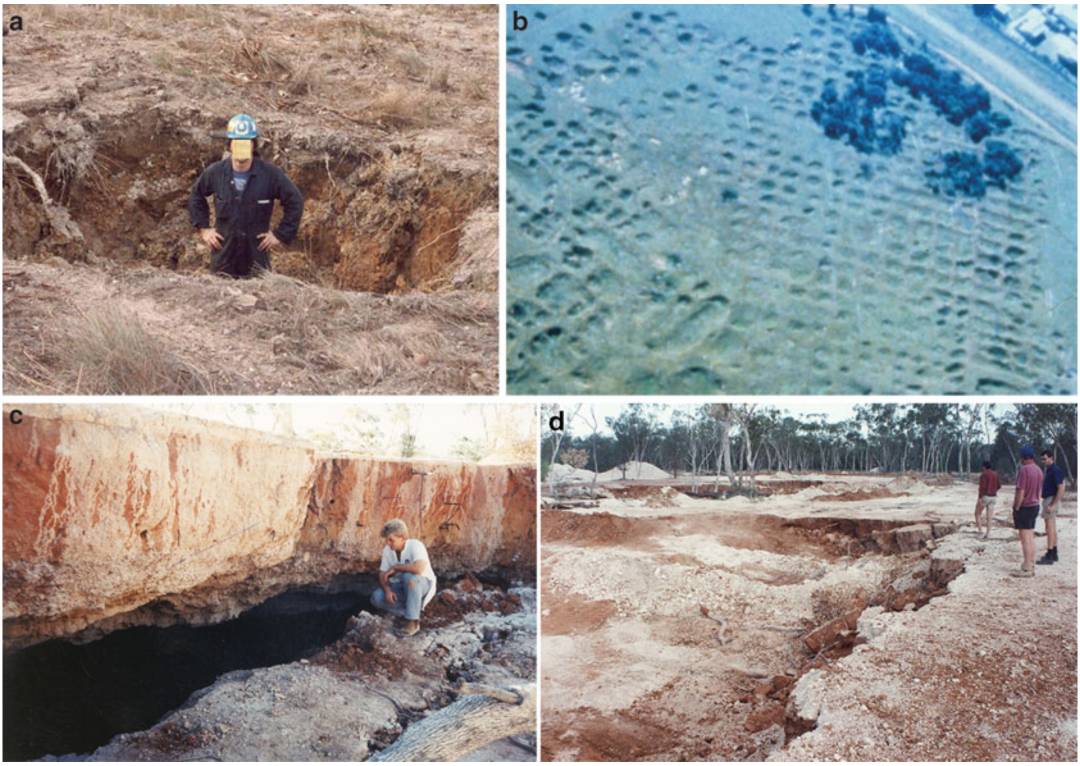


**Fig. 10.13** Examples of sinkhole and plug subsidence events over shallow mine workings. (**a**) An isolated sinkhole above shallow workings. (**b**) An extreme example of sinkhole subsidence in which the roadways have collapsed to the surface. (**c** and **d**) Plug failure over shallow opal mining extraction panels at Lightning Ridge, Australia

Fig. 10.14. This is due to one or a combination of factors that include decrease in effective stress; piping and erosion into the mine workings; increase in surcharge load; and occasionally, lubrication of fracture planes. These effects are more pronounced in alluvium and highly weathered materials.

Often a sinkhole takes the appearance of a small but deep hole on the surface which then enlarges into a conical shaped depression as unconsolidated surface deposits erode into the depression. Its depth can be much greater than the mining height because fallen overburden splays in the underlying workings and in dipping seams can run downhill. Water inflow aggravates these behaviours. More detailed discussions on the mechanics associated with sinkhole formation are provided by Karfakis (1986), Dyne (1998), Canbulat (2003) and Brady and Brown (2006).

Plug failures are more regional in extent than sinkholes as they tend to be associated with en masse caving of large areas of totally extracted workings at shallow depth. These circumstances can give rise to extensive zones of extension above the workings that increase the risk of dynamic collapse associated with shear around the abutments of the plug. Rainfall events can elevate this risk. These aspects are discussed in more detail in Sect. 3.5 in relation to mining at shallow depth.

### 10.4.3 Classical Subsidence Behaviour

The classical model of surface subsidence is based on assumptions that:

- the surface topography is flat;
- the seam is horizontal;

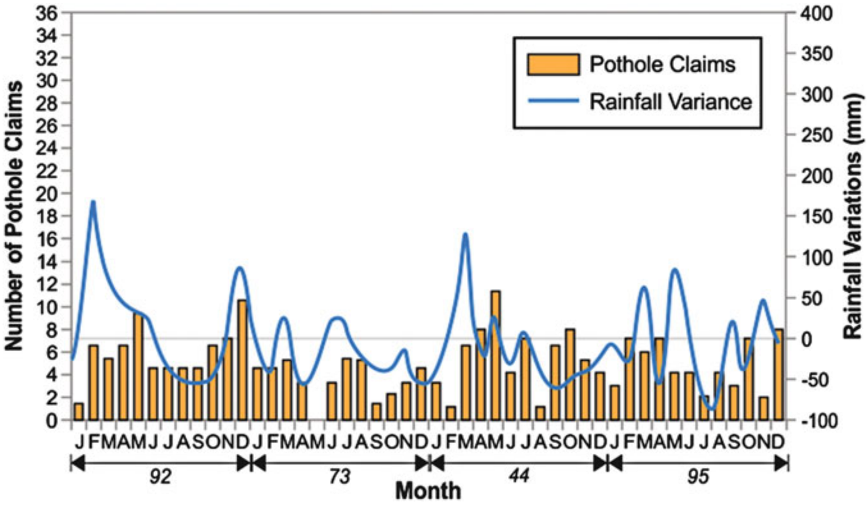


**Fig. 10.14** Correlation between rainfall and sinkhole occurrences in the Newcastle Coalfield of NSW (Redrafted after Cole-Clark 2001)

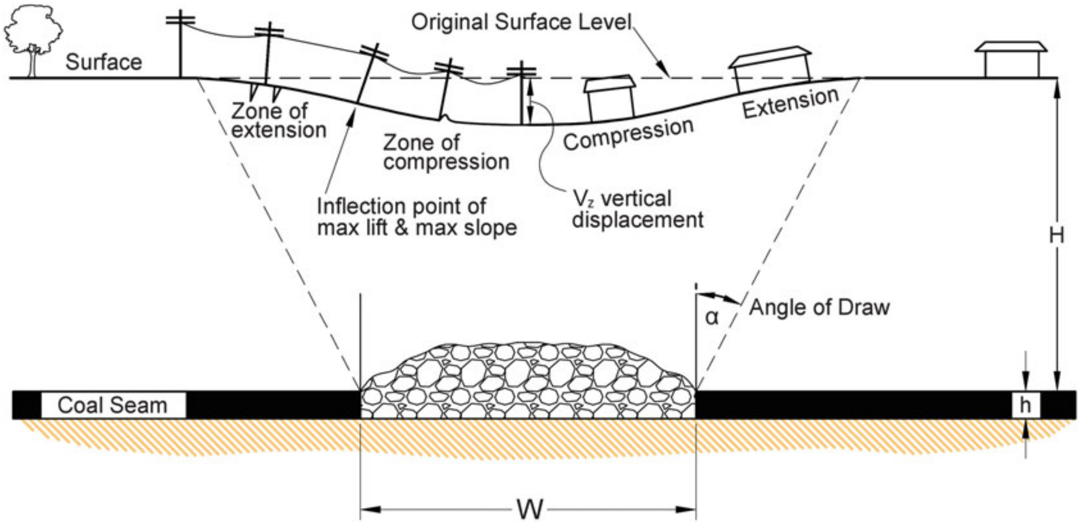

Original Surface Level
Surface
Zone of extension
Zone of compression
Inflection point of max lift & max slope
Compression
Extension
$V_z$ vertical displacement
H
α
Angle of Draw
Coal Seam
h
W

**Fig. 10.15** Exaggerated diagrammatic representation of the components of surface subsidence in a flat topography (Adapted from Galvin and Associates 2004)

- the rock mass is free of geological disturbances; and
- the mine workings are laid out on a regular pattern.

The behaviour of an isolated excavation provides the basis for the model, with surface movements resulting from a combination of displacement of the overburden into the excavation and compression of the excavation abutments under the weight of undermined overburden not supported by the goaf. Compression of the abutments and a horizontal component of displacement towards the subsidence trough results in surface movement extending beyond the footprint of the excavation. The **angle of draw** delineates the limit of mining induced vertical displacement on the surface, as shown in Figs. 10.15 and 10.16 and defined in Sect. 3.3.1. Although as noted in Sect. 3.3.1, the concept of a

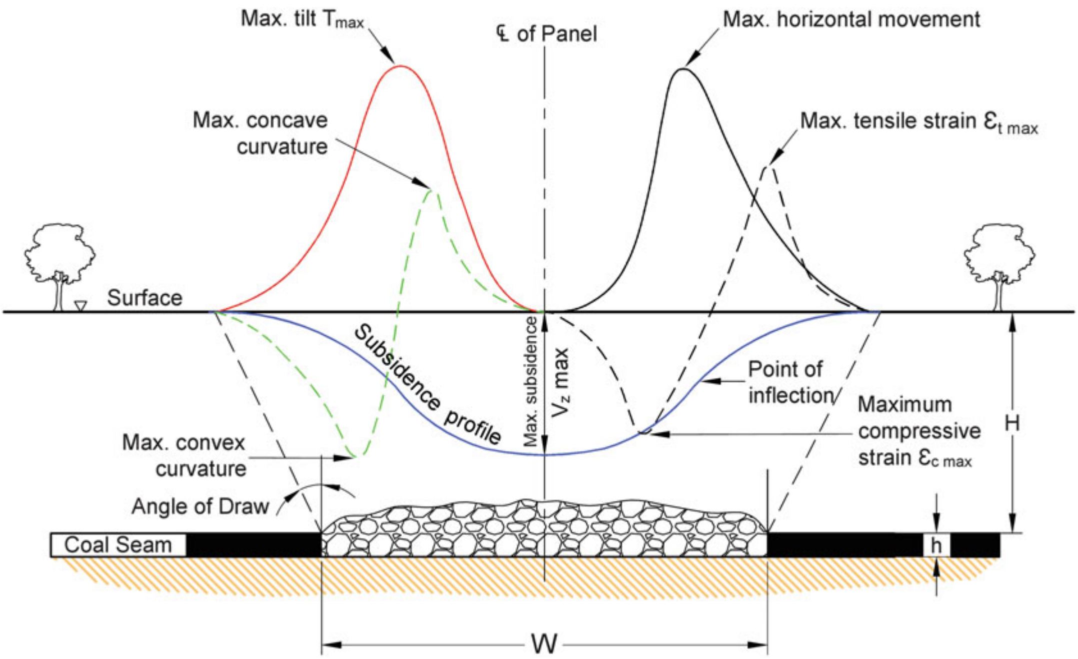

Max. tilt $T_{max}$
℄ of Panel
Max. horizontal movement
Max. concave curvature
Max. tensile strain $\varepsilon_{t\,max}$
Surface
Subsidence profile
Max. subsidence
$V_z$ max
Point of inflection
Maximum compressive strain $\varepsilon_{c\,max}$
H
Max. convex curvature
Angle of Draw
Coal Seam
h
W

**Fig. 10.16** Exaggerated graphical representation of behaviour of surface subsidence components in flat topography (Adapted from NCB 1975)

line demarcating the boundary between moving and stationary ground is theoretically untenable, it is helpful in conceptualising ground behaviour.

Because surface movements can also have natural causes, such as seasonal variations and prolonged dry and wet periods, it can be difficult to identify the lateral extent of vertical movements induced by mining. Therefore, it is standard practice to specify a limiting value for vertical displacement caused by mining. This value is very often taken to be 20 mm, even though it is not uncommon for 50 mm or more of vertical movement to be associated with natural causes. Ideally, the baseline reference points against which vertical displacement can be assessed should be based on at least 24 months of monitoring to better account for seasonal variations in natural ground behaviour.

When the surface subsides in the shape of a trough, it curves outwards near the perimeter of the trough and inwards towards the centre of the trough, shown in a grossly exaggerated two-dimensional manner in Figs. 10.15 and 10.16. This phenomenon is referred to in subsidence engineering as **curvature** and is measured in terms of the inverse of the radius of a circle of corresponding curvature (usually of the order of kilometres).

Curvature in an outwards direction results in the ground stretching, or **hogging**, and is referred to as **convex curvature.** Curvature in an inwards direction causes the ground to **sag** and move closer together and is referred to as **concave curvature.** Implications of this behaviour are:

- Sag within a bed will induce compressive strain in the upper surface and tensile strain in the lower surface, resulting in shearing along bedding planes and fresh fracture surfaces as the overburden bends and subsides.
- Points on the surface move in both a vertical direction and a horizontal direction as they subside into the subsidence trough. Vertical movement is referred to in subsidence engineering as **vertical displacement**, $V_z$. Horizontal movement is broken into two components, being **transverse horizontal displacement**, $V_x$, across the width of a

panel and **longitudinal horizontal displacement**, $V_y$, along the length of a panel.

- When two adjacent points undergo a different amount of vertical displacement, the **slope** of the ground surface between them changes, which then induces **tilt** or **lean** in features located on the surface. Generally in subsidence engineering, slope and tilt are expressed in terms of millimetres of change in distance per metre run, or mm/m. However, from a layman's perspective, it can sometimes be helpful to express these values as a percentage or a ratio. For example, 1 mm/m might be better expressed as 1 in 1000 or 0.1 %.
- Difference in horizontal vectors (magnitude and direction) between two adjacent points produces **horizontal distortion**, $\gamma$, also referred to as **shear strain.**
- Convex curvature induces **tensile strain**, $\varepsilon_t$, which is measured in terms of millimetres of extension per metre length, or mm/m, and has a positive notation. Induced tensile strain results in a reduction in pre-mining compressive stress. Once compressive stress is reduced to zero, further increase in tensile strain is likely to result in the development and/or reactivation of extension cracks within the rock mass.
- Concave curvature induces **compressive strain**, $\varepsilon_c$, which is also measured in terms of mm/m but with a negative notation. Compressive strain can cause surfaces to rupture or buckle and heave.
- At some point on the surface, curvature changes from convex to concave, producing a **point of inflexion** or **transition point**.
- Bending is induced in long surface features that are located in zones of curvature. The **deflection ratio** is a measure of the severity of this bending. It is defined as the inverse of the length of a straight line joining two points on a curved surface, divided by the perpendicular distance (or offset) from the straight line to the curved surface (or chord) at the mid-point of the straight line. That is, deflection ratio equals offset divided by chord length.

As mining approaches a surface feature, the feature will start to tilt towards the excavation. Maximum tilt occurs at the point of inflexion. If the zone of concave curvature then passes beneath the feature, the feature will start to tilt back in the opposite direction and, if the mining area is sufficiently large, it should in theory ultimately return to its original vertical inclination (Figs. 10.15 and 10.16).

The ground surface in the vicinity of a feature displaces horizontally in a similar manner. It initially moves towards the approaching excavation and then, as it subsides into the trough, it starts to move back in the opposite direction, with a component of these movements being directed towards the centre of the subsidence trough (Fig. 10.17). Differences in the direction and magnitude of these horizontal displacements between neighbouring points induces horizontal shear in the intervening ground surface. Research by Li et al. (2011a), suggests that horizontal shear can be a significant factor in small horizontal strain situations associated with small panel width-to-depth ratios, W/H, particularly when mining at depth.

As the edge of an excavation is approached from the solid side ahead of an active mining face, the ground surface experiences stretching and, hence, increasing tensile strain that builds to a maximum value. From that point, there is a gradation from the point of maximum tensile strain, through a point of zero strain at the point of inflexion, to a point of maximum compressive strain as illustrated in Figs. 10.15 and 10.16. The inflexion point moves towards the goaf as panel width-to-depth ratio increases, typically lying over the goaf at W/H ratios greater than 0.5.

Although ground slope and strain are expressed in terms of mm/m, differential ground movements may not be uniformly distributed in this manner in the field. In particular, tensile strain may accumulate at specific cracks or natural joints, with crack width typically ranging from several hundred millimetres at mining depths less than 200 m, to generally hairline at mining depths approaching 500 m in level topography. Additionally, buckling

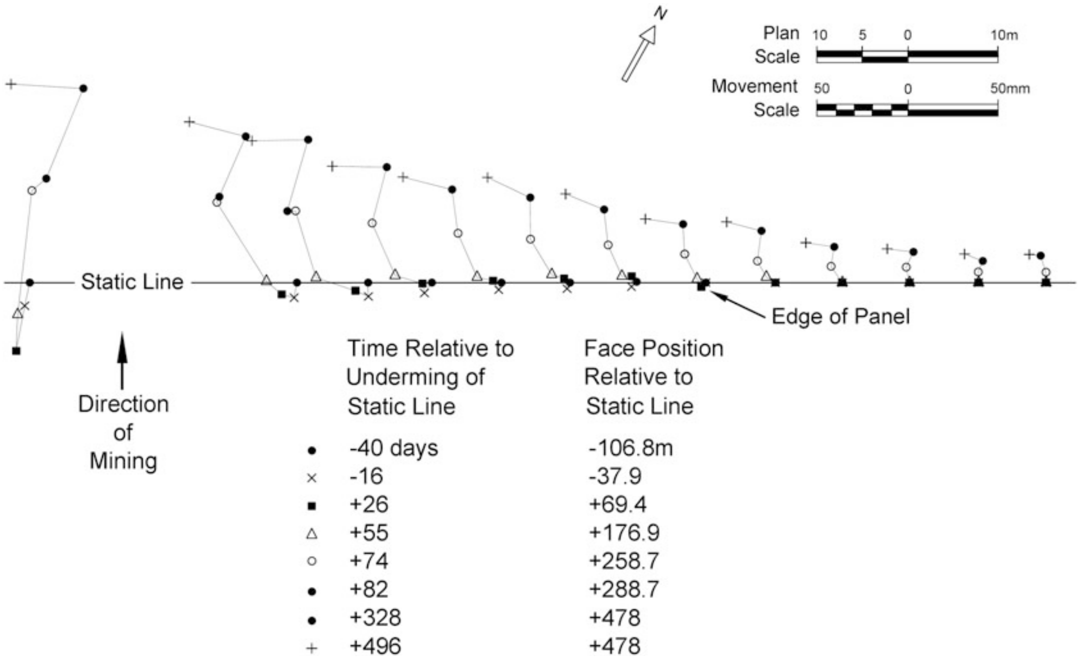


**Fig. 10.17** Measured horizontal displacement with respect to time and space along a transverse line over a longwall panel (After Steijn 1980)

of near surface strata under the effects of high compressive strains can cause cracking which has the appearance of being tensile in origin. This is a localised and superficial effect associated with failure of a thin surface layer of the rock mass, with the deeper rock mass continuing to be subjected to compression.

It is a standard practice in subsidence engineering to express maximum vertical displacement, $V_z$, as a fraction of the mining height, h. This relationship ($V_z/h$) is known as the **subsidence factor**. Figure 3.14 shows subsidence factor plotted against panel width-to-depth ratio for isolated (first) panels in a number of coalfields throughout the world. Bord and pillar mining cases, in which the roadways are very narrow compared to depth, fall at the extreme left of the curve, where subsidence is negligible. The different curves reflect the different geology and stress states in the various coalfields. Creech (1995) and Tobin (1998) report, for example, that higher subsidence occurred over longwall panels in the Newcastle Coalfield, Australia, when they were orientated parallel to the regional fault, dyke and joint directions. Mills (2012) assessed Tobin's database to conclude that the sag component of subsidence increases in the presence of high horizontal stress. The three conclusions are not unrelated as geological structure in this coalfield is generally normal to the principal horizontal stress.

All points on the surface do not experience the full range of subsidence impacts. Depending on their location in the subsidence trough, some points may return to a state of near zero strain, tilt and slope after subsiding into the trough. Others on the flanks of a trough may be left in a state of induced tilt or slope, and tensile, compressive and shear strain. This state may or may not be permanent, depending on whether panels interact and if one or more adjacent panels are subsequently extracted. If the impact is permanent, the consequences can range from negligible to severe, being determined by the magnitude of the subsidence parameters, the nature and position of affected natural and man-made surface features, and the extent and effectiveness of mitigation and remediation measures.

In situations such as shown in Figs. 3.15 and 10.1a where subsidence over each excavation develops virtually independently of that over a previously extracted adjacent panel, over 90 % of the final vertical displacement, tilt and strain at a surface point usually occurs within months of mining. Otherwise, subsidence effects develop and change incrementally with the extraction of subsequent panels, so that a new equilibrium may not be established for several years, as for the example shown in Fig. 3.16. In such cases, the overall vertical displacement profile at any point in time is found by summing the incremental displacement profiles up to that time (Sect. 3.3.1). In the case depicted in Fig. 3.19, for example, vertical displacement continued to increase gradually over Longwall 21b during extraction of at least the next four longwall panels, albeit at a diminishing rate (discussed further in Sect. 10.4.5).

A common characteristic of vertical displacement along the longitudinal axis of a total extraction panel is that it has a steeper slope at the starting end of the panel than at the finishing end. This behaviour tends to be associated with overburden containing strong stiff beds. A range of reasons have been postulated for it that relate to time to failure, exposure to horizontal stress, and panel finish point relative to a periodic weighting cycle (see, for example, Holla 1985; Mills 2012). The extensive occurrence of the behaviour suggests that it may be more to do with differences between the initial failure mode of a plate that is clamped on four sides and the subsequent failure mode of a plate that has one free edge.

### 10.4.4 Site-Centric Subsidence

A range of site-centric behaviours imprint on classical subsidence behaviour, the more common being associated with:

- steep topography;
- massive superincumbent strata;
- valleys, gorges and drainage lines;
- horizontal stress relief;
- dominant regional geological structures; and
- significant changes in depth of cover due to the dip of the seam.

In general, the influence of the first five of these features is greater in high horizontal stress environments.

#### 10.4.4.1 Steep Topography

In steep topography, it is not uncommon for one or more wide open surface cracks to develop towards the top of a hill and for a compression hump to develop towards the bottom of the same hill, with these locations not being in general agreement with the classical model for tensile and compressive strain distribution depicted in Fig. 10.16. Gravity is often advanced as the reason for this subsidence impact, based on the hypothesis that it induces high levels of ground movement in a down-hill direction, thus causing tensile strain to accumulate towards the top of a hill instead of being distributed uniformly down the hill side. While this may be correct, there are other mechanisms that can also contribute to or account for the behaviour. Two of these are classical slope failure triggered by subsidence; and ground distortion caused by different absolute horizontal and vertical displacements and resulting vector movements between the top and the bottom of a steep and high hill.

#### 10.4.4.2 Massive Overburden Strata

Massive, strong strata in the overburden can be capable of spanning many tens to hundreds of metres without failing. Therefore, this strata retards the development of subsidence and modifies the respective contributions of overburden sag and abutment compression to vertical surface displacement. In some cases, uplift of the surface over panel abutments has been associated with massive competent beds spanning across panels (reference, for example, Oravecz 1966; Hardman 1968).

Steps may occur in the subsidence trough as a result of the strata breaking in a cyclic manner as a series of plates, rather than caving in a regular, smooth manner. The steps in vertical displacement give rise to irregular magnitudes and distributions of tilt and strain. If the massive unit is well defined, the situation can be controlled by either making mining panels sufficiently narrow that the massive strata will bridge permanently without generating high abutment stresses, or sufficiently wide that the strata breaks soon after the commencement of mining and at regular and frequent intervals thereafter. At shallow depth, the option may exist in pillar extraction and in some longwall layouts to design panel widths to break the massive strata during extraction of the second or third panel.

#### 10.4.4.3 Horizontal Displacement

It was generally believed up until the mid 1990s that surface displacements due to mining were negligible beyond a distance from the workings of 0.5–0.7 times the depth of mining (corresponding to 26.5° and 35° angles of draw, respectively). However, in the early 1990s it was recognised that horizontal movements were occurring in the vicinity of Cataract Dam, located in the Southern Coalfield of NSW, Australia. The dam wall had been protected from adjacent underground coal mine workings by a 1.5 km buffer (Preston 1992). Extensive longwall mining has taken place at depth in this coalfield, which is noted for its steep and deeply incised topography and for the major and the minor principal horizontal stresses being typically of the order of 2.5 and 1.5 times the vertical stress, respectively. Reid (1998) reported that 'irregular' horizontal movements were detectable well away from mining operations in the coalfield, with these movements appearing to be initiated by mining and to be generally directed towards the centre of the mine subsidence area.

A study by Holla (1997) in the immediate vicinity of longwall panels at shallow depth (~80 m), in low relief topography, identified that horizontal displacement also extended beyond the limit of vertical displacement. Considerably larger horizontal displacements extending further beyond the panel abutments were detected by the study over much deeper workings (450–500 m) in the Southern Coalfield. Hebblewhite et al. (2000) reported measured horizontal displacements in this coalfield in excess of 65 mm towards mine workings that were 680 m away.

Monitoring and research programs, aided by advances in spatial surveying technologies and the relocation of control survey stations further afield, has confirmed that this behaviour is commonly associated with total extraction underground mining methods (Hebblewhite et al. 2000; Reid 2001; MSEC 2007; Li et al. 2011b; Mills et al. 2011). Figures 10.18 and 10.19 show the nature and extent of these movements in the Southern Coalfield, with most of the movement taking place towards the incised valleys (gorges) and active mining areas.

Pells (2011) applied a simple elastic finite element model to study the incremental horizontal displacement associated with extracting a longwall panel in the Southern Coalfield. Figure 10.20 shows contours of horizontal movement as predicted by the numerical model, with bolded values being predicted movements and bracketed values being measured movements at specific points. Similar behaviour is now believed to occur in other coalfields. Mills et al. (2011), for example, measured horizontal movements of 20 mm up-slope and up-dip of longwall workings some 1.6 km away in a lower horizontal stress regime at 210–250 m depth in the Western Coalfield, Australia. Hebblewhite and Gray (2014) attributed far-field horizontal movements and valley closure in the vicinity of Ryerson State Park Dam in Pennsylvania, USA, to longwall mining, with the movements being well outside the confines of the angles of draw predicted by classical subsidence theory.

It is generally agreed that the far-field movements are associated with disturbance of the regional horizontal stress field, although the mechanics are not yet fully understood. It appears that horizontal displacement is comprised of two components, being that associated with strata curvature over extraction panels and that due to stress relief as a result of subsidence related changes in the overall stiffness of the overburden. Horizontal displacements resulting from the latter are primarily a function of the in situ horizontal stress; the direction of mining relative to the direction of this stress direction; the depth of mining; the modulus of

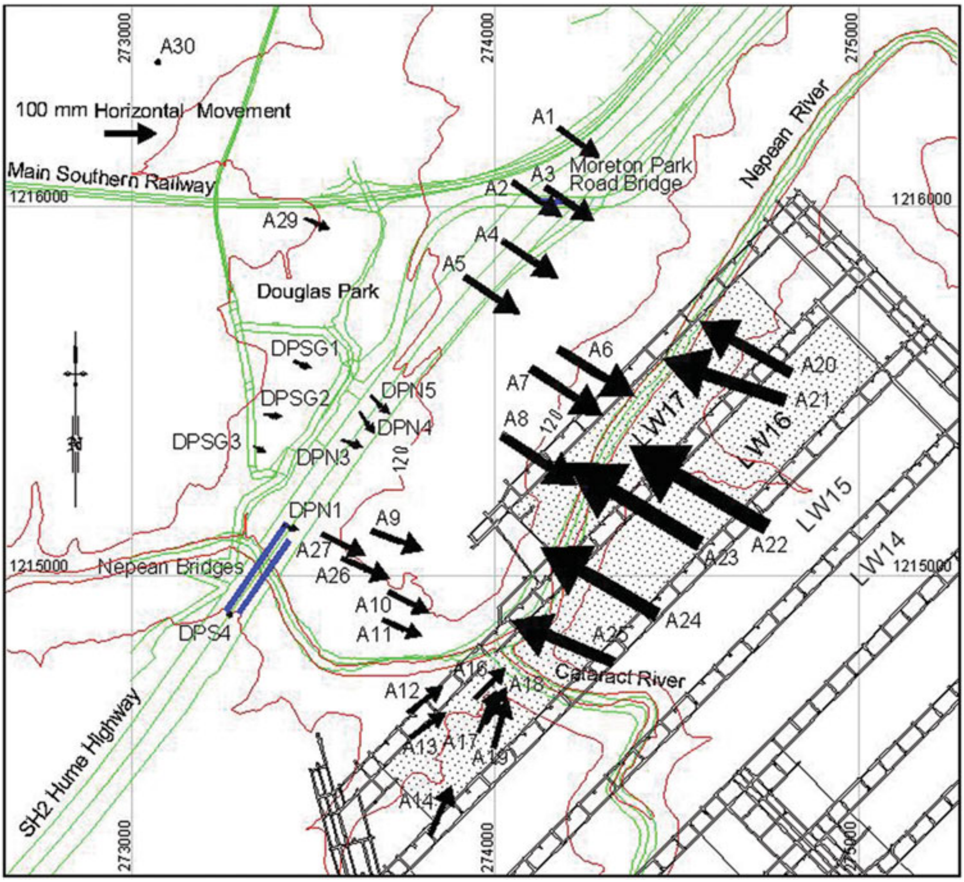

273000
A30
100 mm Horizontal Movement
Main Southern Railway
1216000
A29
Douglas Park
274000
A1
Moreton Park
Road Bridge
A2
A3
A4
A5
Nepean River
275000
1216000
DPSG1
DPSG2
DPSG3
DPN5
DPN4
DPN3
DPN1
120
120
A6
A7
A8
A20
A21
LW17
LW16
LW15
LW14
A9
A27
A26
A10
A11
1215000
Nepean Bridges
DPS4
A22
A23
A24
A25
1215000
Cataract River
A12
A16
A18
A13
A17
A19
A14
SH2 Hume Highway
N
273000
274000
275000

**Fig. 10.18** Valley closure and far-field horizontal movements associated with longwall mining in the vicinity of Cataract Gorge, Australia (After Hebblewhite et al. 2000)

the overburden; and the regional extent of total extraction mining. This component accounts for so-called **far-field movements**, which develop incrementally at a point as mining approaches. The rate of decay of horizontal movements rapidly drops with distance from the excavation. This is reflected in Fig. 10.20, resulting in ground strains being inconsequential.

#### 10.4.4.4 Valleys and Gorges

Steep, incised topography, which is often associated with high horizontal stress fields, interrupts the transmission of horizontal stress, causing it to be re-directed from the hills and into the floor of valleys and gorges as shown in Fig. 10.21. This can lead to overstressing of valley floors, with the near surface rock strata bending and buckling upwards. The valley is deepened over time through natural weathering that sustains this process, which is referred to as **valley bulging**.

Field investigations confirm that valley bulging can result in the creation of voids beneath watercourses, often in the form of open bedding planes which can act as underground flow paths for groundwater and stream water (Patton and Hendren 1972; Fell et al. 1992; Everett et al. 1998; and Waddington and Kay 2002a). The natural underground flow of a stream is referred to as **underflow**. It can occur independently of the surface flow or the two flow paths may intermittently connect.

Secondary extraction causes further disruptions on a regional scale to this incised

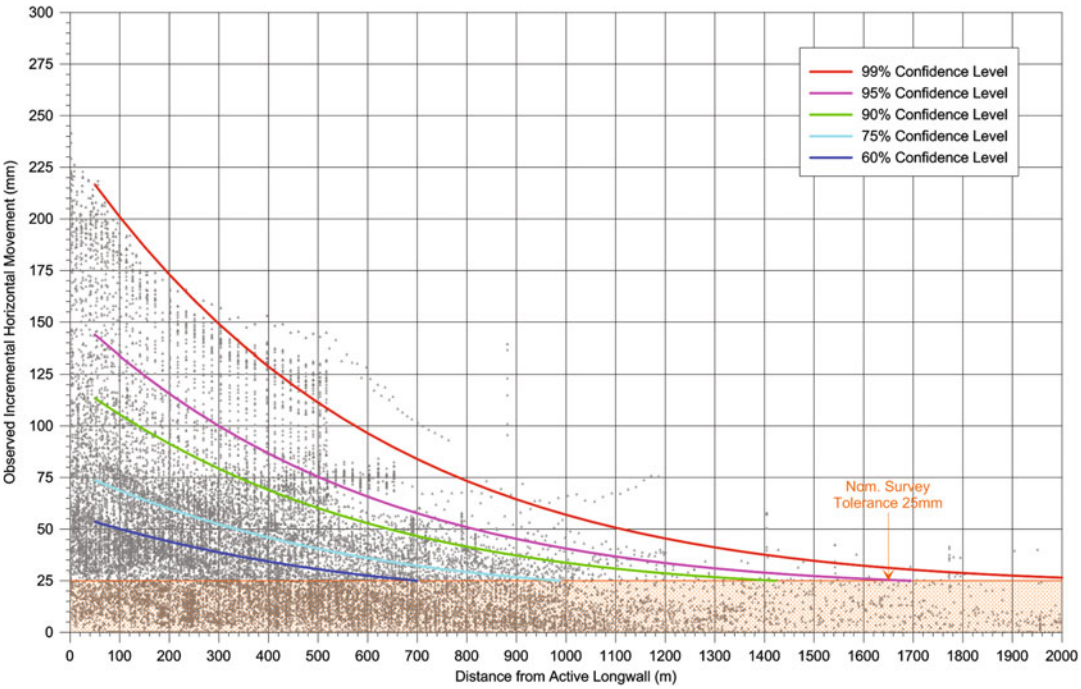


**Fig. 10.19** Magnitude and rate of decay of incremental horizontal displacements with distance from longwall panels in the Southern Coalfield of Australia (After Barbato 2015)

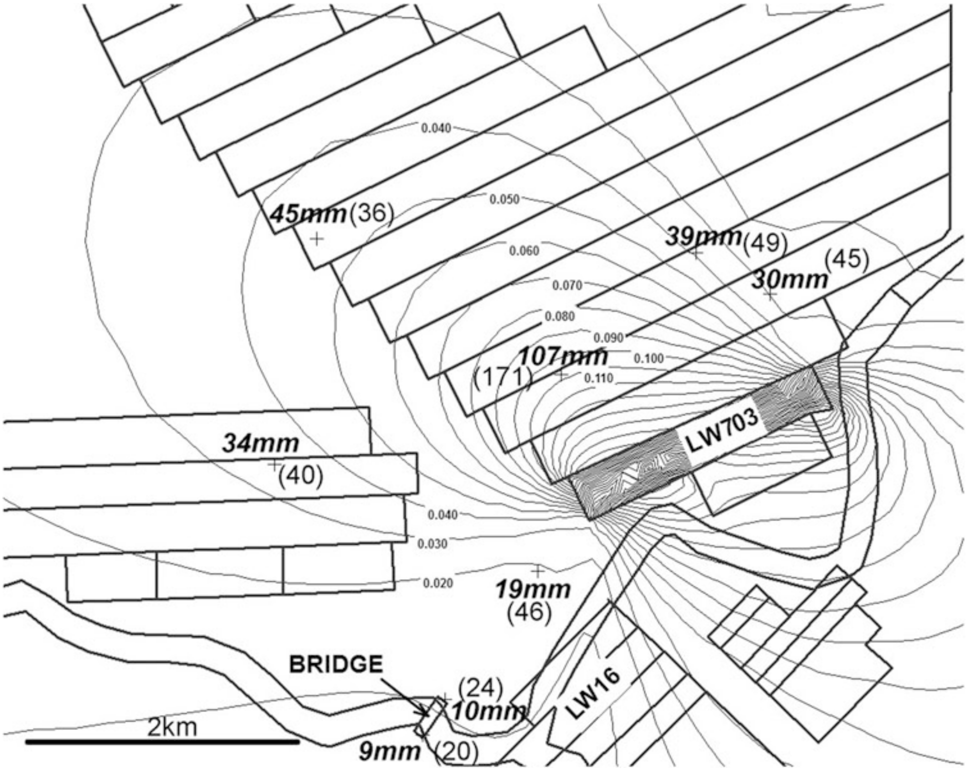

0.040
0.050
0.060
0.070
0.080
0.090
0.100
0.110
45mm (36)
39mm (49)
30mm (45)
107mm (171)
LW703
34mm (40)
0.040
0.030
0.020
19mm (46)
BRIDGE
(24) 10mm
9mm (20)
LW16
2km

**Fig. 10.20** Comparison between predicted (*bold*) and measured (*bracketed*) incremental horizontal movements due to extraction of Longwall 703 (After Pells 2011)

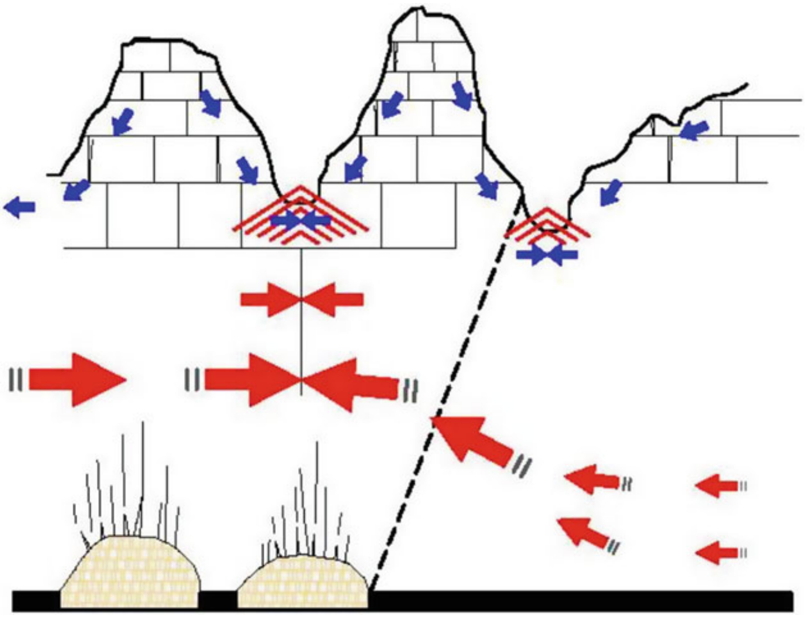


**Fig. 10.21** Conceptualisation of the distribution of primitive (*blue*) and resultant (*red*) horizontal stress above mine workings in steep and deeply incised topography (Modified from Galvin and Associates 2005)

topography and to the natural horizontal stress system by creating soft inclusions comprised of the caved and fractured zones over the excavations (Sect. 5.2.5). These inclusions result in a redirection of horizontal stress into the floor strata of excavations and, depending on the depth of mining, into the upper roof overlying extraction panels. One effect of a valley is to remove confinement to near-surface subsiding strata, allowing it to relax and dilate. These effects individually or collectively can lead to activation of existing shear planes and the development of new shear planes just below a valley floor. This situation may be compounded when mining is conducted within a high horizontal stress field at a depth sufficient to result in a constrained zone. This is because horizontal stress will be redirected through the constrained zone, as shown conceptually in Fig. 10.21, thereby also contributing to an increase in horizontal stress acting across a valley floor.

These disruptions contribute to two ground responses, namely:

- Valley closure, whereby both sides of a valley move horizontally towards the valley centreline. Valley closure is not significantly influenced by the orientation of the valley relative to the mining layout or to the goaf and can develop outside the angle of draw (Hebblewhite et al. 2000; Keilich et al. 2006; Mills 2007; and Mills 2011).
- Uplift of the valley floor due to valley bulging, resulting in buckling, shearing and overriding of the valley floor and near surface strata. The difference between the amount of vertical displacement that could have been anticipated in the absence of a valley and that which eventuates is referred to as **upsidence**. In some instances, upsidence can result in the final absolute level of a valley floor being higher that that prior to undermining, in which case it is sometimes referred to as **uplift**.

Features of upsidence and valley closure are:

- Both behaviours can extend up to several hundred metres beyond the angle of draw. Waddington and Kay (2002b), for example, report that while upsidence in the base of Cataract Gorge above Longwall 8 at Tower

Colliery in NSW was 350 mm, the upsidence in the cliff lines was around 100 mm and the upsidence effect extended for a distance of some 300 m on each side of the gorge.

- The movements develop incrementally with each panel extracted.
- The magnitudes of the incremental movements increase with increase in incremental vertical displacement.
- Both valley closure and upsidence are often greater in the presence of a headland (valley spur).
- The behaviours can also be associated with gentle valley systems and creek beds, albeit that the magnitudes of the closure and upsidence movements are less.
- Effects and impacts are a function of the bedrock lithology (including composition, thickness, and the nature and dip direction of bedding) and jointing (Kay et al. 2011).

Hence, the ground movements that occur around excavations are complex and may include classic subsidence ground responses to mining; elastic ground movements associated with redistribution of horizontal stress on a regional scale; gravity induced unravelling; and localised buckling and shear failure. It is difficult to identify the individual contributions of these components. Some components may even operate simultaneously in opposite senses. For example, an area could be subjected to downwards vertical displacement at the same time that it is being subjected to upwards valley bulging.

If upsidence occurs within the angle of draw of the mine workings, ground movements due to classic subsidence can also contribute to buckling and shear in the near-surface strata, thus generating an extensive network of fractures and voids in the valley floor. The formation of an upsidence fracture network has been monitored in detail for a number of years at Waratah Rivulet, above the workings of Metropolitan Colliery, Australia, using an array of surface and subsurface instrumentation (Mills and Huuskes 2004; Mills 2007). This has provided considerable insight into how upsidence develops incrementally and how the associated fracture network becomes deeper with the passage of adjacent longwalls. Ultimately, in this particular case, the main fracture network extended to a depth of about 12 m and bed separation to a depth of some 20 m as shown in Fig. 10.22. Studies have also revealed that besides upsidence extending for tens of metres laterally beneath valley sides, it may also not follow the line of the valley floor. Rather, it can cut across valley headlands and bends in the valley.

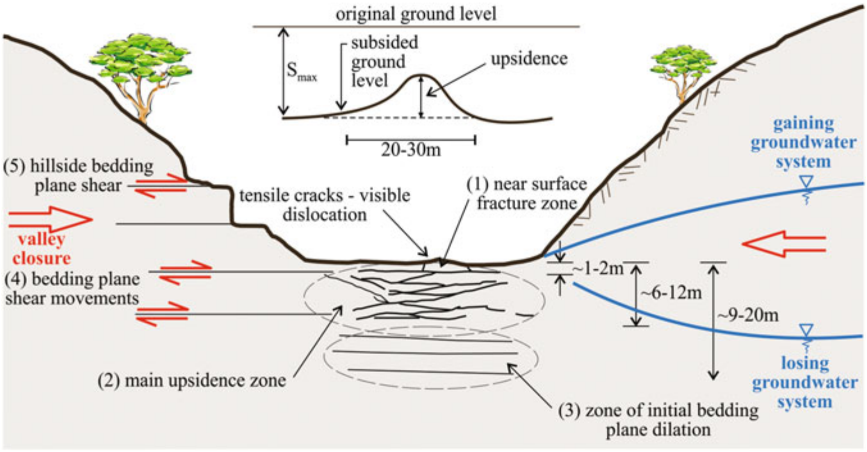

original ground level
$S_{max}$
subsided ground level
upsidence
20-30m
(5) hillside bedding plane shear
tensile cracks - visible dislocation
(1) near surface fracture zone
gaining groundwater system
valley closure
(4) bedding plane shear movements
~1-2m
~6-12m
~9-20m
(2) main upsidence zone
(3) zone of initial bedding plane dilation
losing groundwater system

**Fig. 10.22** Upsidence fracture network determined from surface and subsurface monitoring at Waratah Rivulet, Metropolitan Colliery, Australia (After Mills 2007)

## 10.4.5 Prediction of Classical Surface Subsidence

### 10.4.5.1 Techniques

Surface subsidence prediction techniques can be classified as empirical, analytical, numerical, and hybrid. Empirical techniques are premised on the back-analysis of field performance and, therefore, reliability of outcomes is dependent on the size and representativeness of the database. The more common empirical techniques are:

1. Graphical: This involves plotting suites of curves showing relationships between various parameters and subsidence outcomes. There may be no engineering basis for some of the relationships. The Subsidence Engineers' Handbook produced by the British National Coal Board (NCB) in 1965 and updated in 1975 (NCB 1975) is a well known example of a graphical prediction approach. Predictions are specific to British conditions, with the approach proving unreliable when applied to South African and Australian conditions.
2. Upper Bound. This involves predicting maximum subsidence values on the basis of upper bound envelopes which cover the majority of measured data points pertaining to a specific parameter. Examples of this approach are those of Holla (1985, 1987). The methodology is not so much concerned with predicting subsidence outcomes at specific locations or with producing profiles of the various subsidence parameters across the subsidence trough, but more with restricting subsidence to less than designated maximum values at all sites under all circumstances.
3. Profile Function. This technique attempts to define the shape of the vertical displacement curve by an equation. This equation is then mathematically differentiated to produce a profile of tilt. In turn, the tilt profile is differentiated to produce a profile of curvature. Calibration factors derived from back analysis of field data are then applied to the curvature profile to produce predictions of strain. The methodology is confined in general to single (isolated) excavations as it cannot replicate non-symmetrical subsidence profiles.
4. The Incremental Profile Method (Waddington and Kay 1995; MSEC 2007) is a form of profile function that accounts for how the profile of vertical displacement varies with superincumbent strata stiffness. The approach is based on 'reverse engineering' the subsidence prediction process by utilising large databases of subsidence information to define the characteristic shape of each increment of vertical displacement associated with extracting a series of panels. These are summed to produce a vertical displacement profile, from which tilt, curvature and strain can be calculated in the same manner as that described for the profile function technique. While the concept has been known for a long time, it has only come to find wide application since 1994. This prediction technique offers a number of benefits over many other empirical techniques because variations in depth, seam thickness and dip can be taken into account and subsidence predictions can be produced at any nominated point on the surface.

A range of prediction techniques based on analytical and numerical methods find application in subsidence engineering (for example, Kratzsch 1983; Coulthard and Dutton 1988; Heasley 1998; Salamon 1991; Alejano et al. 1999; and Keilich 2009). The attributes of these types of prediction methods are discussed in general in Sect. 2.7. They can be particularly apt for the prediction of subsidence if applied sensibly. A field performance database is required for calibration purposes and, therefore, predictions need to be accepted with caution at greenfield sites. The reasonably accurate prediction of vertical and horizontal surface displacement is a fundamental, but not exhaustive, test that an analytical or numerical model is simulating rock mass behaviour in a sensible manner.

Swarbrick et al. (2014) provided an overview of numerical approaches to the prediction of surface subsidence, their strengths and limitations and how emergent technologies may be able to overcome some or all of these limitations. They concluded from comparative studies of different numerical modelling techniques that:

- There are several different mechanistic modelling approaches that appear to have the capacity to provide acceptable subsidence predictions when compared to standard empirical methods.
- These predictions are sensitive to material properties and, in some cases, small changes in input can result in significant changes in output.
- While predictions may fit observed subsidence well, they may be poor predictors of other subsidence related impacts such as strains.
- Mechanistic models that incorporate anisotropic behaviour and the development of anisotropic behaviour, in particular, appear to be more reliable subsidence predictors.
- Each different modelling technique appears to require its own set of calibrated parameters in order to give the best predictions when compared to standard empirical methods.
- Mechanistic approaches may produce counter-intuitive responses when extrapolated to configurations significantly beyond the calibration database.
- Caution is required whenever mechanistic models are used to provide predictions outside the calibration database. This includes the prediction of sub-surface cracking, changes in hydraulic conductivity and surface tilts and strains.

Hybrid prediction techniques are based on the application of analytical and numerical techniques supported by back-analysis of field data. The Influence Function technique is one of the more popular hybrid techniques, although it is sometimes classified as an empirical technique and sometimes as an analytical technique. Influence function methods predict vertical displacement at a point by superimposing and summing the individual subsidence troughs at that point produced by each infinitesimal element of extraction at the seam horizon. The element directly under the surface point makes the most contribution to vertical displacement at the point, with the contribution of neighbouring elements dissipating with lateral distance from this point.

Once a vertical displacement profile has been generated with an Influence Function, tilt and strain can be calculated by mathematically differentiating this profile in the same manner as described for the Profile Function technique. The Influence Function technique can be applied to a wide range of mining layouts but the selection of the appropriate influence function is dependent on the availability of field data. The reader is referred to Kratzsch (1983) and Whittaker and Reddish (1989) for more detailed information on the technique.

### 10.4.5.2 Reliability

#### Angle of Draw

Vertical displacement is an effect of mining but not a measure of the impact of mining on surface features. In an elastic environment, vertical displacement theoretically extends to infinity while, in reality, the distance that vertical displacement is detectable from an excavation is a function of the precision of the surveying technique. Hence, it is not feasible to define protection zones around structures on the basis of a zero vertical displacement criterion. Rather, protection zones have come to be defined by bounding curves of radius, R, centred on principal points of the surface structure as shown in Fig. 10.23, with mining prohibited or restricted within the zone delineated by these curves. Historically, this radius was prescribed to be a fraction of depth of mining, based on local experience of subsidence impacts on features. For example, a radius of H/2.7, or 0.35H, was prescribed in South Africa (Salamon and Oravecz 1976).

A radius of protection around a structure can also be expressed in terms of an angle measured from the vertical. For example, a radius of 0.35H corresponds to an angle of approximately 20°. This angle should not be confused with an angle of draw. Angles of draw are subject to considerable fluctuations and variability, with primary

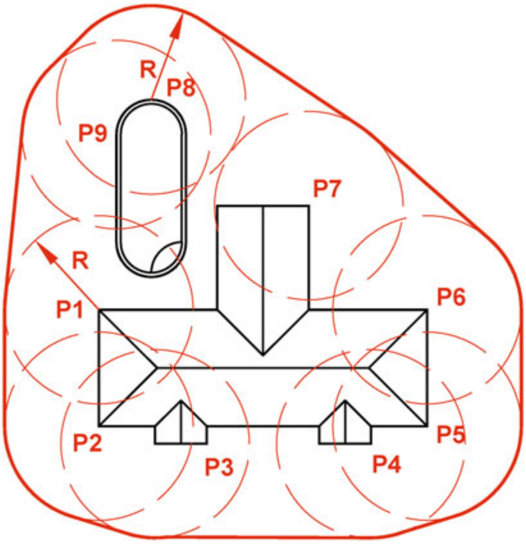


**Fig. 10.23** Method for delineating a protective pillar to restrict subsidence impacts on a surface feature

sources being seasonal ground movements; seam dip; topography; the nature of the superincumbent strata; the stiffness of the loading system; and the abutment stress profile around an excavation. The last three of these factors are interactive and can give rise to significant differences in the angle of draw between the starting end, finishing end and side flanks of a panel. In general, angle of draw is less at shallower depths of cover.

O'Rourke and Turner (1981) recorded a range in angle of draw of 16–45° associated with select mining operations in the UK, Europe and the USA, from which they concluded that angle of draw decreased with increase in the strength of overburden rocks. Holla and Barclay (2000) recorded a range in angle of draw of 2°–56° across an overall panel width-to-depth ratio, $W_o/H$, range of about 0.25–4.25 in the Southern Coalfield of NSW, Australia. The average of the 74 measurements was 29°, with nearly 70 % of the observed values being below 35°. MSEC (2012) recorded a similar range in values in this coalfield, shown in Fig. 10.24 for individual panels with a width-to-depth ratio, W/H, ranging up to about 0.7. The mean value of the measured angles of draw was 26°. In both cases, values were based on a cut-off vertical displacement of 20 mm. These ranges in angle of draw are due in part to the fact that when panel width is subcritical at depth, angle of draw can change as additional panels are extracted.

It is not uncommon for mine design to be based on an angle of draw of 26.5°, which is a convenient value because it equates to an offset distance from the edge of the mine workings of half the depth of mining (0.5H). Field experience in Australian conditions indicates that this value is generally conservative for critical and supercritical panel width situations. However, in the case of subcritical to critical panel width-to-depth ratios, it is likely to correlate poorly with measured values, as reflected in Fig. 10.24. This does not automatically mean that subsidence presents a threat to a surface feature when the angle of draw is greater than the design value, since angle of draw is a measure of a mining-induced effect and not a mining-induced impact. High angles of draw at low panel width-to-depth ratios or at large mining depths can be associated with very low vertical displacements and, more importantly, with negligible differential movements. However, to the general public and to regulators with a layman's understanding of subsidence, such high angles of draw can appear alarming.

Hence, it is advisable not to use angle of draw as the sole design basis for protecting surface structures against mining impacts, unless these impacts can be confidently correlated against angle of draw. A protective zone that can be defined independently of angle of draw may be preferable, provided that there is a basis for relating subsidence impacts to the width of this zone. Another approach which has found widespread application in assuring the protection of a critical surface feature (such as a dam wall or very significant archaeological heritage) is to place a buffer zone of a designated width around the feature and then calculate the angle of draw from the extremity of that buffer (see for example, Reynolds 1976).

### Vertical Displacement

Irrespective of the subsidence prediction technique employed, vertical displacement is the only component of surface subsidence that is

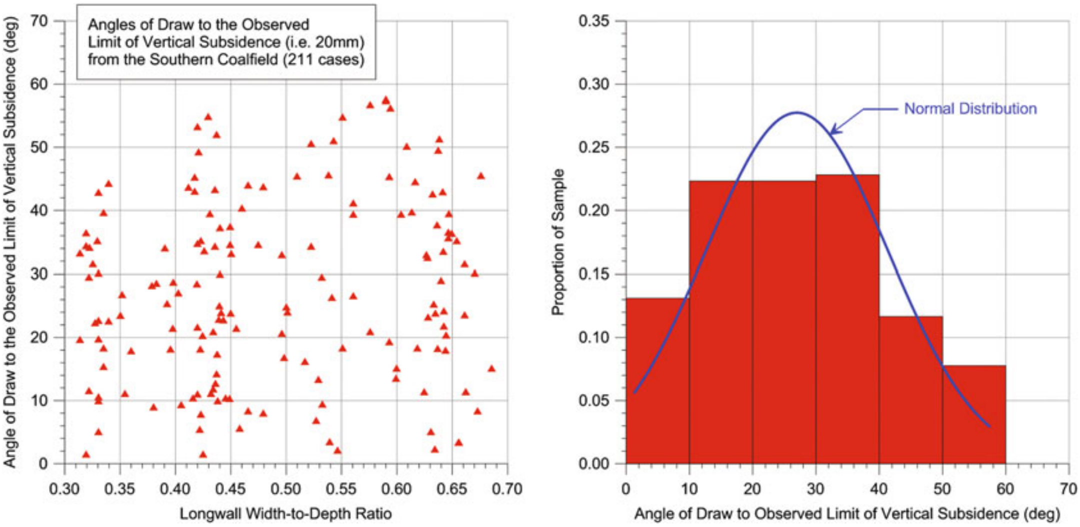


**Fig. 10.24** An example of angle of draw plotted against individual panel width-to-depth ratio for the Southern Coalfield of Australia (After MSEC 2012)

predicted directly. Tilt is predicted by either differentiating the vertical displacement profile or by multiplying the vertical displacement profile by a calibration factor. Similarly, curvature is calculated by differentiating the tilt profile or by multiplying the vertical displacement profile by a calibration factor. Therefore, any error in the prediction of vertical displacement can carry over to tilt and curvature predictions.

The upper bound approach and the incremental profile method find extensive application in predicting vertical displacement in Australia. By definition, the upper bound approach is likely to over-estimate vertical displacement in many situations. The incremental profile method has a tendency to over-predict total vertical displacement because a conservative approach was adopted when constructing the incremental vertical displacement profiles (MSEC 2007). Over-prediction of vertical displacement does not necessarily mean that associated subsidence parameters will also be over-predicted. This is because these parameters are based on the rate of change of vertical displacement, rather than the absolute value of vertical displacement. Nevertheless, it is generally the case that tilt and strain are also over–predicted.

A number of empirical and analytical subsidence prediction techniques are capable of producing reasonably accurate predictions of vertical displacement, typically within ±150 mm. The more noteworthy of these are the incremental profile method, the influence function method and a number of numerical modelling codes. Figure 10.25 shows the level of accuracy that can be associated with the two dimensional version of Salamon's laminated model (Quinteiro and Galvin 1995). Nevertheless, due to the site-specific nature of subsidence behaviour, significant deviations from predictions occur occasionally as a result of an unexpected change in geology or geotechnical conditions. Creech (1995), Waddington and Kay (2001b) and Gale and Shepherd (2011) provide insight into these types of occurrences.

### Tilt, Strain and Curvature

The prediction of tilt and, in particular, strain is more problematic. In the first instance, strain predictions are often made with respect to axial and longitudinal survey lines when, as evident from Fig. 10.17, maximum strain may occur in a different direction. Inaccuracies in measuring these parameters can contribute to variance between predicted and measured values. Because tilt and strain vary across the subsidence trough, measured values are a function of **bay length**, being the distance between the survey points on

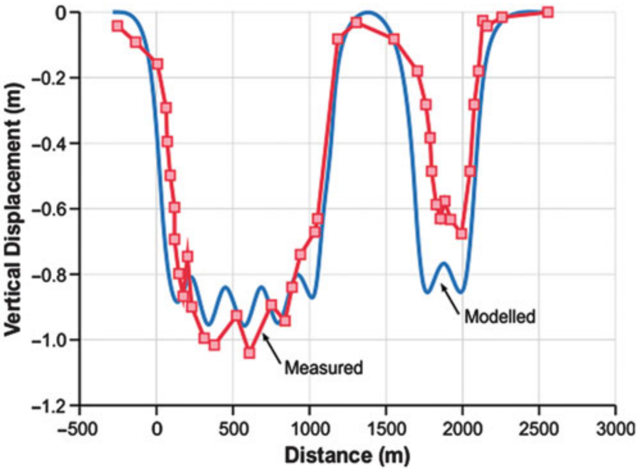


**Fig. 10.25** Measured vertical displacement at a deep Australian colliery compared with that predicted using a two-dimensional version of Salamon's laminated numerical model (After Quinteiro and Galvin 1995)

which the calculations are based. If bay length is too great, calculated values of tilt and strain may be significantly less than field values over shorter distances within a bay. Variance also arises when tilt and strain are concentrated at specific locations rather than being uniformly distributed. Therefore, bay length should always be quoted and the distribution of tensile cracks and compression humps should be reported when presenting with tilt and strain measurements. A common standard is to set bay length at 1/20th of the depth of mining.

If overall outcomes are based on summing individual bay length outcomes, then careful consideration should be given to the potential for, and impact of, cumulative errors. Check survey runs based, for example, on every sixth survey station can provide an effective means of gauging accuracy in the case of survey lines that are straight.

There is no direct correlation between vertical displacement or curvature and strain. Hence, all strain prediction methods rely on some form of calibration or correlation factor. One method that has found widespread application is to predict maximum strain by multiplying the ratio of maximum vertical displacement to depth, $V_{z\ max}/H$, by an upper bound calibration factor that varies with panel width-to-depth ratio, W/H (reference, for example, NCB 1975; Holla 1987; and Holla and Barclay 2000). Another simple and popular method that enables strain to be predicted continuously over a panel is based on assuming that strain is a fixed multiple of curvature. For example, it has been common for strain predictions to be equated to 10 times curvature in the Northern Coalfield and 15 times curvature in the Southern Coalfield of Australia.

An analysis of field data by Barbato (2015) has identified a number of limitations with this latter approach. In particular, the accuracy of the procedure is affected by:

- survey order of accuracy;
- site-centric subsidence behaviour, such as valley closure;
- strain associated with stress relief; and
- the method of calculation, with curvature being calculated over three survey marks (two bay lengths) while strain is calculated over two survey marks (one bay length).

Figure 10.26 partially reflects some of these limitations. The grey diamonds are based on the measured curvature and strain at a point, i.e. the strain measured in each survey bay plotted against the average of the curvatures at each end of the survey bay. This has been a common method of presenting curvature and strain data. However, these points can be misleading, as both the measured curvature and strain profiles are very irregular, meaning that large strains can have small corresponding curvatures (i.e. local curvature) while the overall/regional curvature is

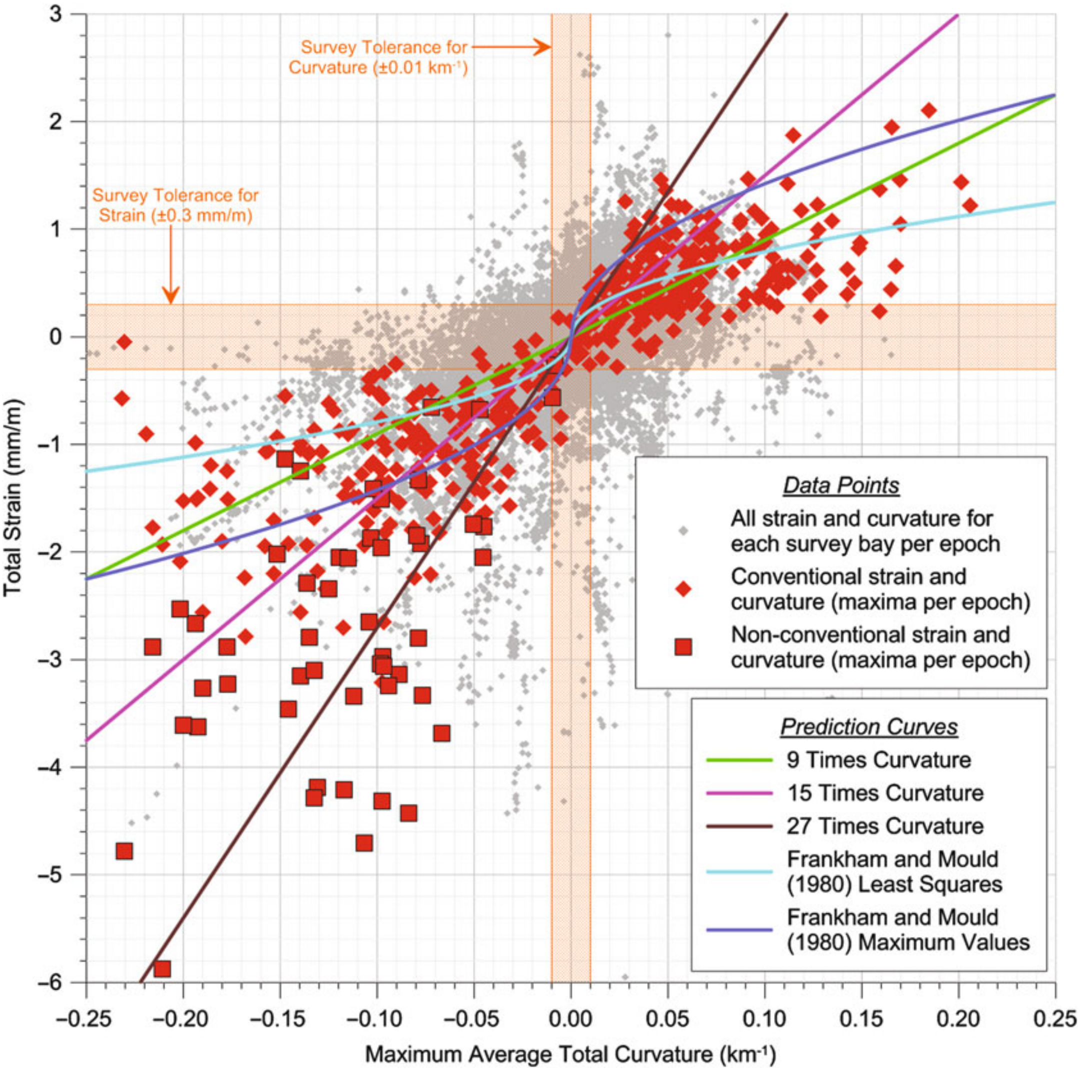


**Fig. 10.26** Various correlations between curvature and strain based on measurements made during the extraction of longwall panels in the Southern Coalfield of Australia (After Barbato 2015)

much larger. This is especially true in locations of irregular movement where large positive and large negative curvatures can occur in adjacent bays, resulting in a meaningless averaged local value.

The red diamonds and squares in Fig. 10.26 are based on comparing the maximum strains with the maximum overall/global curvature within each zone, regardless of whether these maxima coincide. This provides a better guide to the relationship between the overall/global curvature and strain. The various prediction lines between curvature and strain shown in the figure are based on the red diamond and square data points.

A particular difficulty in predicting strain is that, as shown in Fig. 10.27, it is not uncommon for measured surface strains to be in the opposite sense (tensile or compressive) to that predicted by classical subsidence theory, albeit that the overall type of strain is generally consistent with classical subsidence theory. This behaviour is attributed to localised 'skin' effects associated with factors such as cross bedding and buckling

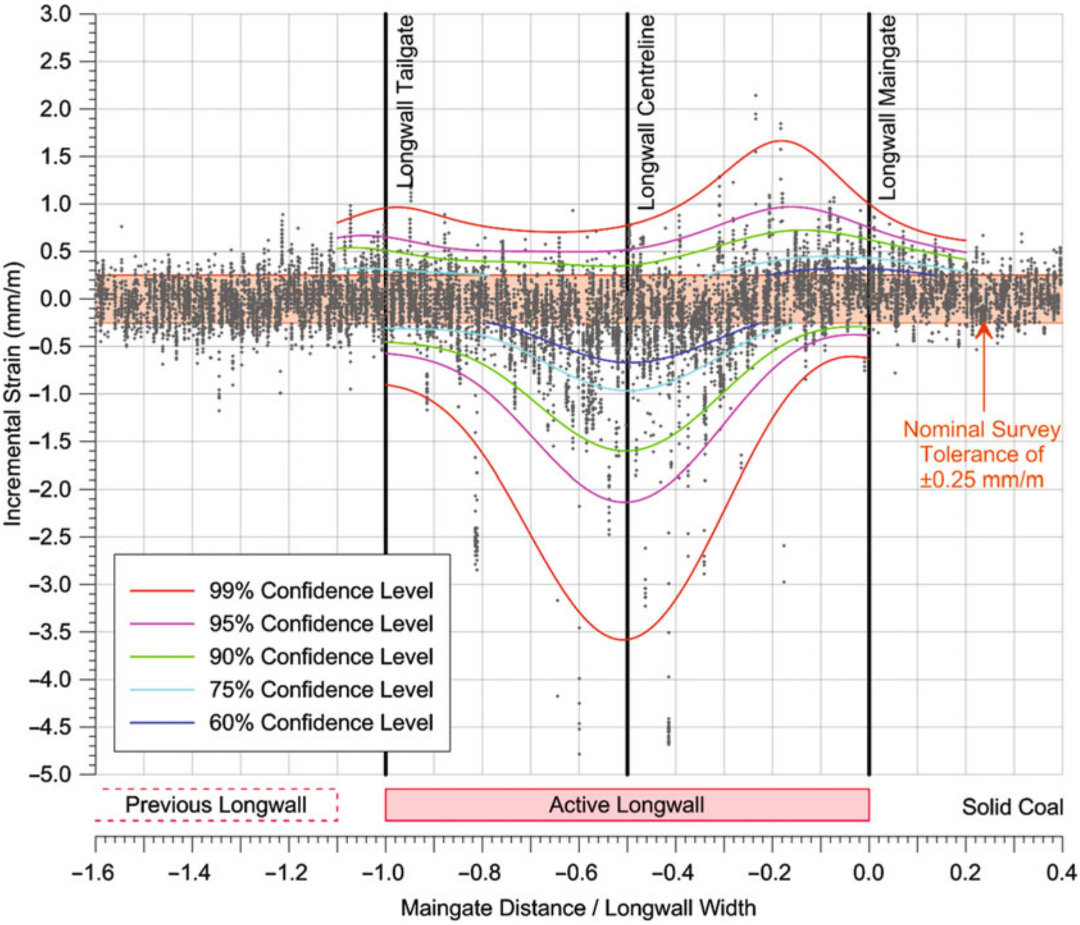


**Fig. 10.27** An example of the distribution of measured ground strains relative to a subsidence trough showing that while the general trend in strain behaviour is consistent with theory, strains can be opposite to this trend on a localised basis (After Barbato and Sisson 2011)

failure of thin beds of strata. It is also influenced by pre-existing natural jointing, depth of bedrock and surface slope. A subtle and important point to note is that a measured tensile strain can be an expression of a reduction in compressive strain, with the ground continuing to be in compression. An elevated in situ horizontal stress regime complicates the determination of the state of stress in the ground on the basis of strain measurements.

Hence, irrespective of what mechanistic based technique is used to predict strain, predictions will be unreliable to some degree. This may be managed in some circumstances by subjecting databases such at that shown in Fig. 10.26 to stochastic analysis in order to assign confidence levels to predictions. Barbato and Sisson (2011) report on this approach.

In assessing the accuracy of tilt and strain predictions, it is important to examine the accuracy of the profile of vertical displacement. Poor correlations between predicted and measured tilt and strain are sometimes due to the actual vertical displacement profile being displaced laterally relative to the predicted profile. Impacts of this translation in surface profile can be catered for by extending the assessment zone around a structure by a given amount, typically 20 m, and basing subsidence predictions for the feature on worst case values within that zone.

Prediction techniques for horizontal shear arising from differential strain, or distortion, across a structure are in their infancy. Li et al. (2011a) have proposed an approximate methodology based on angular distortion.

#### 10.4.5.3 Multiple Seams

The prediction of subsidence above panels in multiple seam situations is dependent on a range of factors that include the extent of superpositioning of the workings, type of mining method and nature and thickness of the interburden. Hence, generally this is a matter that needs to be addressed on a site-specific basis. However, if the total extraction panels in adjacent seams are both of critical width (see Sect. 3.3.1) and the parting thickness is not excessive, typically less than 30–50 m, there is a range of field experience of incremental vertical surface displacement associated with extraction of the second seam approximating 90–100 % of the mining height in that seam (see Sect. 5.4).

#### 10.4.5.4 Magnitude, Rate and Duration of Subsidence

The magnitude, rate and duration of subsidence are a function of many variables that include the type of mining method; the extraction height; the in-panel percentage extraction; the depth of mining; the rate of mining; foundation behaviour; the width of individual extraction panels; the overall width of a series of mining excavations; and the type of mine workings in adjacent seams. In the case of bord and pillar workings in which all elements of the pillar system are stable (being the coal pillar, roof and floor interfaces, and roof and floor foundations as described in Chapter 4), vertical displacement is typically in the range of a few millimetres to 50 mm. In the case of stable partial extraction layouts, vertical surface subsidence typically ranges up to 300 mm but can be more. This subsidence usually develops at rates of less than tenths of millimetres per hour, and often less than hundredths of millimetres per hour. As such, both the rate of and magnitude of vertical displacement are rarely discernible on the surface.

This situation can change if any of the elements of the pillar system deteriorate over time or if the pillar system fails suddenly. Sudden failure results in an instantaneous step increase in vertical displacement. Otherwise, pillar spall and foundation failure result in a gradual increase in vertical surface displacement, which develops into an instantaneous increase if, ultimately, the pillar system fails suddenly. Foundation failure associated with creep is a classic form of pillar system failure in which vertical displacement may increase over many years, initially accelerating and then decelerating. This is exemplified in Fig. 10.28, which shows the rate of development and duration of vertical displacement associated with foundation failure of a tuffaceous claystone floor during and after the formation of partial extraction workings at a depth of 160 m in the Lake Macquarie region of NSW, Australia. At one stage, vertical displacement increased at a rate of almost 90 mm/month over a 3 month period.

There are a number of instances in the same mining region that show vertical surface displacement increasing over more than a 20 year period, albeit at very low rates. A feature of all these cases and others such as that reported by Ditton and Sutherland (2013), is that they are associated with high percentage extraction, partial pillar extraction layouts that involve wide excavation spans in weak floor environments. In these situations, pillar strength is secondary to foundation bearing capacity, for which the design knowledge base is still rudimentary and incomplete (see Sect. 4.8.3 and Appendix 4). The design of long-term stable partial pillar extraction workings is deceptively complex and experience would suggest, problematic, once percentage extraction rates exceed about 60 %, other than in strong roof and floor conditions.

In longwall mining, the rate of vertical surface displacement is primarily dependent on extraction height, rate of mining, panel width-to-depth ratio and the nature of the superincumbent strata. Extraction height determines the amount of surface displacement and the rate of mining determines the time available for this displacement to develop. Panel width-to-depth ratio

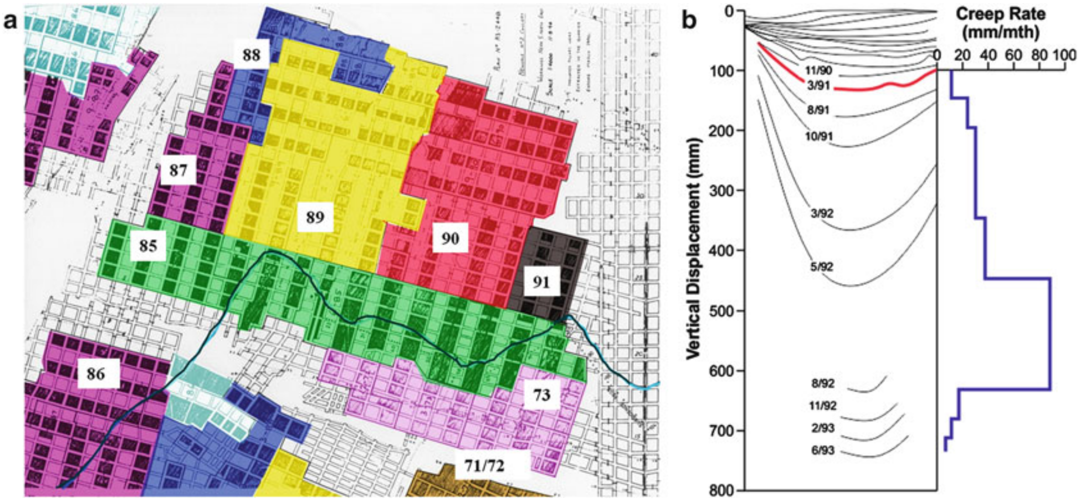


**Fig. 10.28** An example of the rate of development and magnitude of vertical surface displacement due to bearing capacity failure of the floor associated with partial pillar extraction workings (panel and pillar mining) at a depth of 160 m (After Galvin 2002). (**a**) Plan showing annual extent of partial pillar extraction operations. (**b**) Plots of magnitude and rate of vertical surface displacement along shoreline; that is, over workings extracted in 1985

determines whether subsidence over a panel develops largely independently of that over adjacent panels. If it does (as shown in Fig. 3.15 for example), then it is generally agreed that about 90 % of the final vertical displacement, tilt and strain at a surface point develops within 3–6 months of mining. This figure is obviously also sensitive to the rate of mining. Otherwise, when surface subsidence develops incrementally with the extraction of subsequent panels (as shown in Fig. 3.16, for example), it may be several years before 90 % of the final movements develop at a surface point. In deep situations, the rate of vertical surface displacement can be in the order of millimetres per day, while at shallow depth it can exceed several hundred millimetres per day, once again depending on extraction height and rate of extraction. An exception arises when bridging or 'hung up' strata breaks and subsides, in which case several hundred millimetres of vertical displacement may occur over a period of minutes.

The remaining 10 % or so of vertical displacement is usually referred to as **residual subsidence**. As reference to publications such as Kratzsch (1983) and Whittaker and Reddish (1989) highlights, this final displacement can develop over a period of months to years, depending on local circumstances. If local conditions remain reasonably constant, it is possible to develop site-specific formulations to predict rate of settlement, examples of which are presented in Kratzsch (1983).

In all situations, consideration needs to be given to the impact on surface subsidence of flooding of the mine workings in the long term. On the one hand, flooding can promote long term stability because of the hydrostatic confining pressure it provides to the mine workings, which acts to unload pillars and to increase pillar strength. On the other hand, flooding can reactivate consolidation of the goaf and promote disintegration of the pillars (see Sect. 11.6), both of which can result in an increase in surface subsidence. This could be in the form of a sudden step increase in vertical surface displacement if the pillar system were to fail.

A common problem when assessing the stability of mine workings in the long term is the loss of survey stations with the passage of time. The maintenance of at least some survey stations can be invaluable in determining the merits of claims of subsidence-induced damage to structures in time to come. They can clarify whether damage is due to mine subsidence or, as often is the case, to other factors such as reactive clays, tree roots or changes in drainage paths.

## 10.4.6 Prediction of Site-Centric Subsidence

### 10.4.6.1 Valley Closure and Upsidence

A mechanistic understanding of valley closure and upsidence is still evolving, with field studies and research concentrated in the coalfields of NSW, Australia, where the phenomena were first recognised. Up until 2014, the prediction of closure and upsidence was most often based on an approximate methodology developed by Waddington and Kay (2002b) that is underpinned by a large empirical database. The methodology is based on a profile of **equivalent valley depth** that takes into account valley depth and shape and applies upper bound prediction curves to measured versus predicted closure and upsidence values, shown in Fig. 10.29. This approach generally produces conservative outcomes, with upsidence and closure being over-predicted in more that 95 % of cases (Kay et al. 2011). As such, it really represents one of 'restricting' upsidence and closure to worst case values, rather than 'predicting' actual outcomes.

The prediction methodology was updated in 2014, with the number of variables on which the empirical methodology was based increased from 4 to 11 by including seven additional geological and topographical factors (Kay and Waddington 2014). The methodology continues to be based on data sourced from the Southern Coalfield of Australia. Hence, the developers caution that although the method can be used in other coalfields, great care needs to be taken to allow for the classical components of horizontal movement across valleys and for the different geological environments and other site-specific circumstances.

Keilich (2009) utilised the Distinct Element code UDEC™ to develop an alternative methodology for predicting subsidence parameters about valleys undermined by an isolated longwall panel (single panel). The modelling suggested that valley closure and upsidence is primarily a function of ground curvature.

Upsidence does not correlate well with closure. This is due to factors such as difficulties in measuring upsidence accurately (for reasons noted earlier) and to different amounts of upsidence being induced by the same amounts of closure, depending on rock type (for example, sandstone, shale, or claystone).

A review of the database by Kay et al. (2011) concluded that closure is a more reliable parameter than upsidence to indicate the extent of valley movements and to assess valley impacts. This is because:

- the observed upsidence movements are very dependent on the locations of survey pegs, which can be outside the point of maximum upsidence;
- the observed upsidence can be underestimated if the survey line across the valley is too short;
- closure is a measure of macro valley movements and, therefore, displays less variation between adjacent valley cross-sections; and
- upsidence is a measure of micro valley movements in the base of a valley and, therefore, is very susceptible to local variations in near surface geology and topography.

### 10.4.6.2 Far-Field Movements

Because negligible strains are associated with far-field horizontal displacements, the impacts of this subsidence parameter are generally considered to be of little consequence (with the exception of valley closure as already discussed). Therefore, the prediction of far-field horizontal movements has not assumed a high importance to date. The apparent stress relief nature of the movements suggests that numerical modelling may be particularly apt for prediction purposes and, in fact, a number of proprietary simulations by consultants have produced reasonable correlations. The simple two-dimensional elastic finite element model of Pells (2011) used to produce Fig. 10.20 was based on adjusting the effective modulus of the overburden to simulate

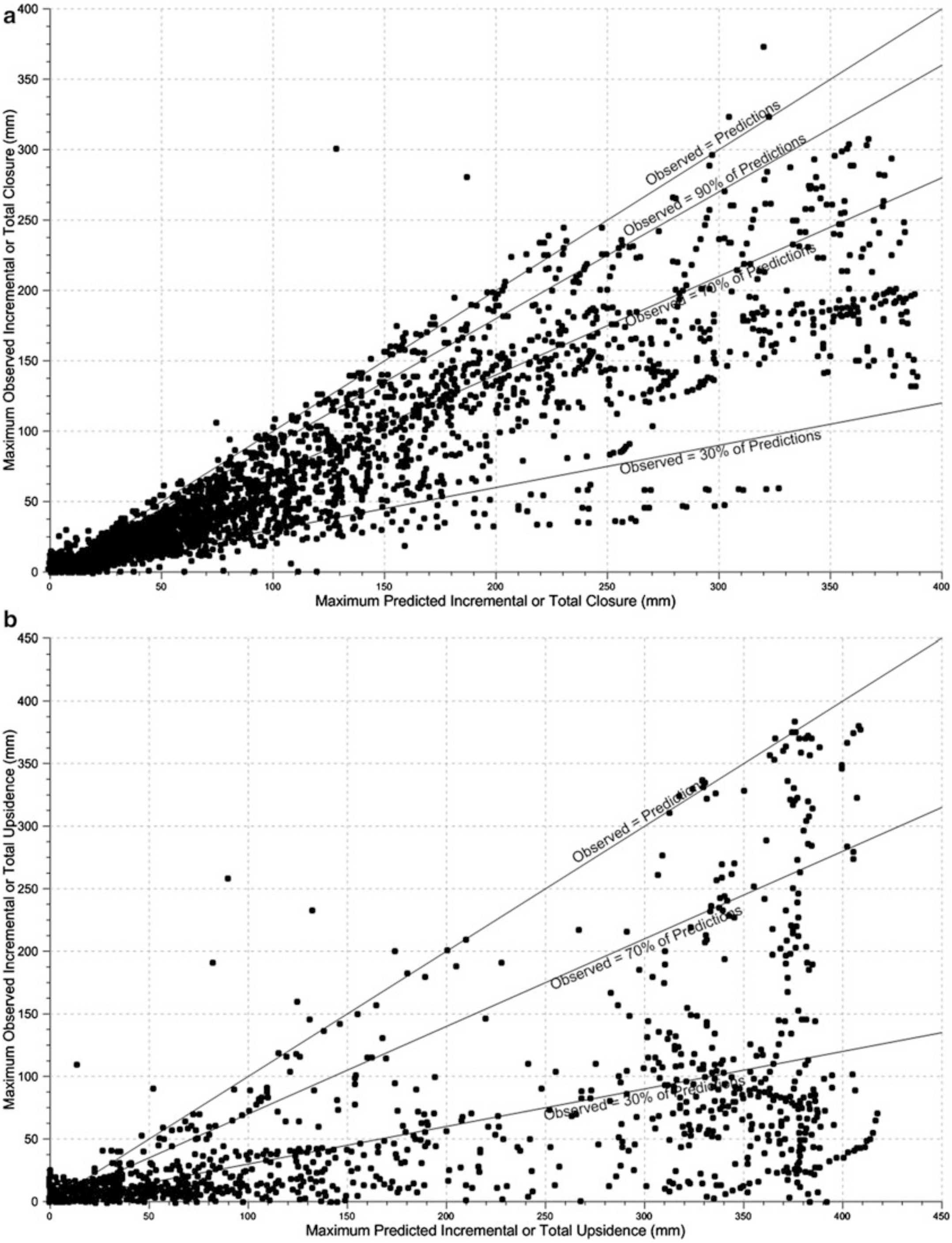


**Fig. 10.29** Comparison between predicted and measured closure and upsidence based on the prediction methodology of Waddington and Kay (2002a) (After Kay et al. 2011). (**a**) Predicted versus measured closure. (**b**) Predicted versus measured upsidence

a soft inclusion created by the caved and fractured zones. It simulated some of the deeper gorges in the area but not geological structures. In most cases, predicted movements were within 80 % of measured, indicating the potential for utilising numerical modelling for making predictions of far-field movements.

## 10.4.7 Surface Subsidence Impacts

### 10.4.7.1 Generally

The impacts of surface subsidence on natural and man-made surface features are determined primarily by:

- site-specific and regional subsidence induced changes in the ground surface. The principal changes relate to vertical position, horizontal position, tilt, strain, curvature, and permeability;
- the number of subsidence cycles to which a feature is subjected;
- the characteristics of the coupling between the ground and the feature of interest;
- the nature and fabric of the feature of interest; and
- the type and effectiveness of mitigation and remediation measures.

Given the variable and interactive nature of these factors, it is not possible to consider surface subsidence impacts only in terms of the subsidence mechanism in play or only in terms of the nature of the feature likely to be affected by subsidence. Rather, impacts have to be assessed on a site-specific basis. Therefore, the following impact descriptions are of a more general nature, are not exhaustive and should not be assumed to apply to all situations. More extensive information in regard to natural features is to be found in DoP (2008, 2010a).

### 10.4.7.2 Sinkholes and Plug Failures

Sinkholes present significant hazards to the general public, mine employees and security of the mine. They have consumed residential and other surface features and equipment operating on the surface, with associated loss of life. Persons falling into sinkholes have also succumbed to noxious and irrespirable atmospheres. Sinkholes provide a potential conduit for air to enter mine workings and promote spontaneous combustion. This can be a very serious problem in old mines that are not segregated by barrier pillars and in areas where extensive surface infrastructure overlies the old workings. While classical subsidence tends to develop gradually, sinkholes often appear suddenly and with no warning whatsoever.

Plug failures can also present significant hazards to the general public and mine employees. In particular, the risk to mine workers of windblast and entombment in the underground workings is greater than for sinkholes.

### 10.4.7.3 Built Environment

Impacts on the built environment are primarily associated with tilt, strain and curvature. Tilt related impacts can include reversal in the flow direction of gravity dependent systems such as gutters and sewers; jamming of windows; and gravitation of doors so that they open or close of their own accord. In extreme cases, structures may slide on their foundations. Changes in tilt are particularly obvious in swimming pools.

In addition to giving rise to strain, curvature may result in one or more ‘hinge points’ developing within a structure. One dimensional strain impacts, which place a structure in extension and/or in compression, include cracks in brickwork and masonry, crushing of brickwork, and uplift of pathways. Horizontal shear induced by differential strain causes structures to shorten in one diagonal direction and extend in the other. This can be reflected in en echelon fracture surfaces in masonry and slip on damp courses.

Although tilt does not usually result in structural damage, it can affect serviceability. Salamon and Oravecz (1976) tabulated four categories of allowable tilt and strain in Poland, with the highest being 15 mm/m of tilt and 9 mm/m of strain. Australian Standard *AS2870-1996 Residential Slabs and Footing* suggested that local deviations in horizontal or vertical slope (tilt) of more than 1 in 150, or 6.7 mm/m, are undesirable and that deviations greater than 10 mm/m will normally be clearly visible. Generally, structural damage is due primarily to strain, curvature and horizontal shear. Because ground movements occur in three dimensions, twisting may also be a contributory factor.

The type of construction is very significant in determining structural tolerance to subsidence effects, including sinkholes, and the feasibility of remediating their impacts. An extensive knowledge base in this regard resides with the NSW Mine Subsidence Board (MSB), established in 1928 following a number of subsidence events in the Newcastle Coalfield, Australia (Cole-Clark 2001). The MSB provides design guidelines for construction in coal mining districts which aim to prevent damage of a structural nature and ensure that infrastructure remains in a state that is safe (no danger to users), serviceable (available for its intended use) and repairable (damage repaired economically). As a general guideline, buildings have been considered to remain safe, serviceable and repairable if tilt is less than 7 mm/m and the level of damage is no worse than Category 2 of AS2870-1996 (Waddington et al. 2011). When using tilt as an impact assessment criterion, one should be alert to the fact that the structure may have not been constructed level in the first instance.

Historically, mining approvals and design guidelines were focused on strain and tilt effects. For example, the damage classification scheme published by the UK National Coal Board in 1975 and summarised in Table A12.1 and Fig. A12.1 of Appendix 12, was based on extension due to ground strain (NCB 1975). These types of approaches have limitations, especially since the location and magnitude of actual ground strains can differ significantly from that predicted and because ground strains may be only partially transferred into structures.

Since the mid 1990s, guidelines for construction in mine subsidence prone districts have evolved, driven by advances in the subsidence engineering knowledge base and construction materials, techniques and standards. It appears that portions of the NCB classification system were adopted in Australian Standard *AS2870-1996 Residential Slabs and Footings* (Waddington et al. 2011). This standard defined five impact categories of structural damage to buildings on the basis of crack width and was supported by a similar system for tilt. However, while AS2870-1996 identified a site class “P” as being relevant to areas susceptible to mine subsidence, the designer was faced with considerable difficulty in identifying an appropriate design protocol (Appleyard 2001). This led to the MSB introducing graduated design guidelines in 2001, presented in Table A12.2 of Appendix 12.

A benefit of using crack width as the main criterion for subsidence impacts is that it provides a clear objective measure by which to classify impact. However, experience has shown that it is a poor measure of the overall impact and extent of repair to a structure, although it should still be used to assist classification (Waddington et al. 2011).

Waddington (2009) researched this aspect in a study of 1037 houses and civil structures located over active longwall workings of Tahmoor Colliery in NSW, supported with 542 historical records, to produce two alternative impact classification systems. The second scheme, presented in Tables A12.3 and A12.4 of Appendix 12, is recommended for general use (Waddington et al. 2011). It is based on the probability of damage and the nature and extent of repairs. The classification system recognises that there is often a difference between the extent of damage from a structural or physical perspective and the extent of repairs that may be necessary. It is reported to better reflect the impacts on a whole building rather than concentrating on crack width. The research concluded that curvature appears to be the most effective subsidence parameter upon which to develop a method of impact assessment, with frequency of impacted structures increasing as curvature increases.

#### 10.4.7.4 Surface Watercourses

Surface watercourses are vulnerable to impacts from both classic and site-centric subsidence. Direct impacts of classical surface subsidence

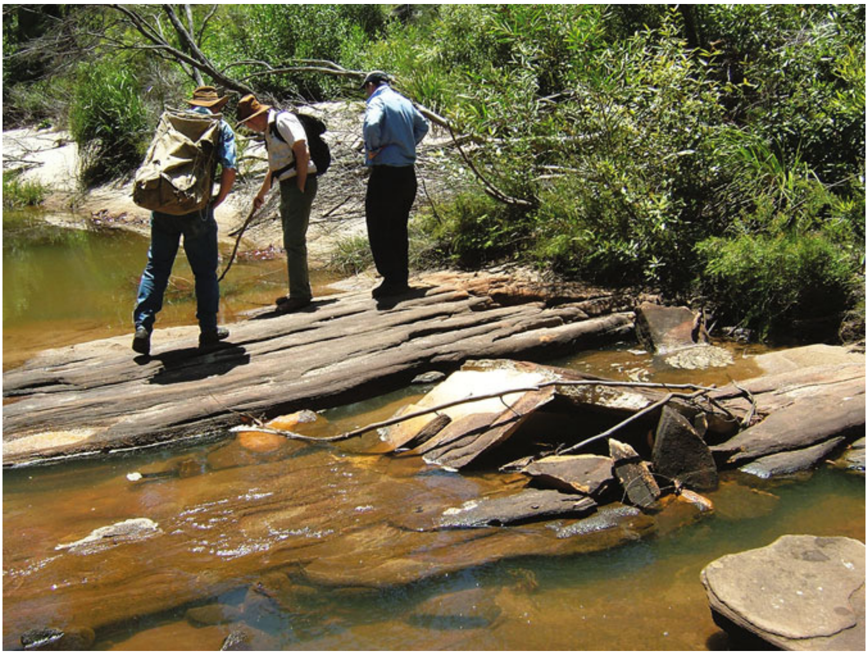


**Fig. 10.30** Buckling of near surface strata in a watercourse subjected to upsidence and valley closure

on watercourses can include lowering of stream embankments; change in stream gradient; tilting of the stream bed so that flow is biased to one side of the watercourse; cracking of the stream bed; and the creation of ponds. In turn, these changes have the potential to impact on the breadth of the stream, including inundation of adjacent land; frequency of flooding; erosion of stream banks; fauna and flora; water quality; archaeological features; and aesthetics and amenity.

Matters that should be considered when evaluating the likelihood, extent and severity of these impacts include natural stream gradient; base flow quantities and velocities; composition of stream banks and bed; and orientation of the watercourse relative to the layout of the mine workings.

Site-centric subsidence adds another dimension. In particular, valley closure and upsidence can cause bedding plane separation, extensive vertical fracturing, and shearing and buckling of stream bedrock, with associated rock debris. Figure 10.30 show an example of valley closure-induced buckling in the bed of a watercourse.

Buckling and fracturing provide an opportunity for some or all of the stream flow to be diverted into a freshly created sub-surface fracture network from where, depending on mining dimensions and hydrogeological conditions, water may re-emerge downstream of the affected area, escape to another catchment, or flow into the mine. Although flow re-emerges downstream of the upsidence affected area when a constrained zone is present, it has yet to be categorically confirmed that subterranean water flow is not diverted laterally through dilated upsidence zones into other catchments. Other subsidence impacts include leakage of rock bars that retain upstream pools and infiltration of water into, or drainage of water out of, groundwater systems.

Environmental consequences can include visible fracturing of bedrock within the bed of the watercourse; physical dislocation of slabs of rock from the bed and subsequent transport and downstream breakdown of these slabs into finer

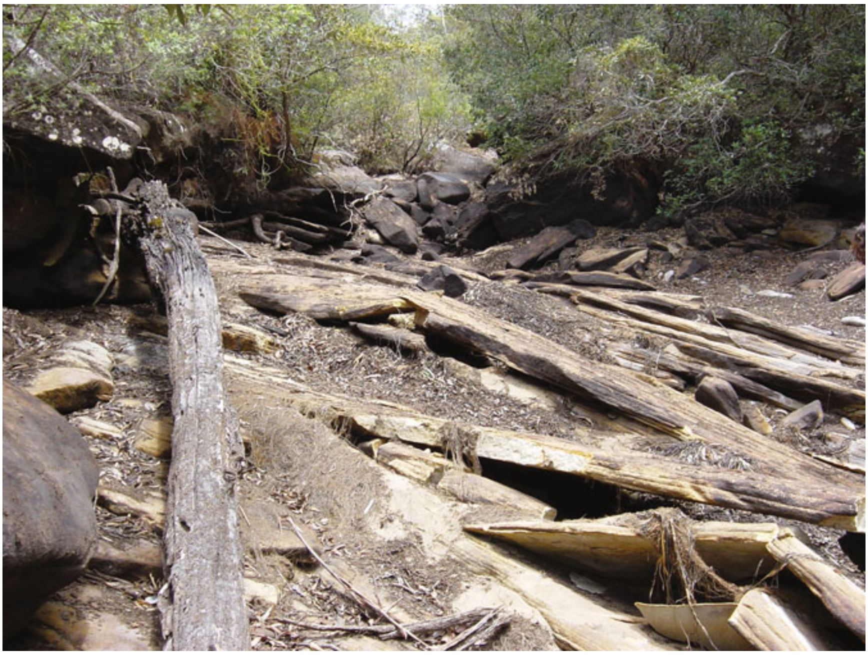


**Fig. 10.31** Drainage of a pool into a subsurface fracture network generated by upsidence as evidenced by buckling of strata comprising the bed of the watercourse

material; loss of water from pools; loss of low flows from the bed of the watercourse; and increased concentrations of dissolved metals, leading to surface iron staining, iron floc matting, algal growth and discolouration of water in pools. Some of these consequences are illustrated in Figs. 10.31 and 10.32.

Site-specific features influence the severity and permanence of valley closure and upsidence related impacts and consequences for surface watercourses. These include:

- The nature of the near surface rock. Laminated and cross bedded strata are more prone to buckle and shear than massive thick strata.
- The width of the valley floor. With all other factors being constant, the severity of buckling and shearing is less in narrow valleys.
- The nature of the watercourse. Perched watercourses are prone to drying out if cracked, while water flow and quantity remain unaffected in drowned valley watercourses.
- The sediment load in the watercourse. High sediment loads are conducive to self-healing of fracture networks.
- The gradient of the watercourse. High gradients result in scouring of the fracture network and, therefore, are not conducive to self healing.

The preceding discussion of the impacts of classical and site-centric subsidence on watercourses is based primarily on experience in the Southern Coalfield, Australia, with more detailed information provided by Waddington and Kay (2002a), Mills and Huuskes (2004), Galvin and Associates (2005), Mills (2007) and DoP (2010). However, the impacts and consequences are not unique to this region, with Stout (2004), Bartsch et al. (2005) and Hebblewhite and Gray (2014) providing insight into experiences in the USA.

#### 10.4.7.5 Valleys

In addition to being associated with upsidence, valley closure has the potential to impact on the stability of the valley sides, including local features such as caves and, in particular, on any rigid structures that span the valley. There is a history of damage to structures that span valleys

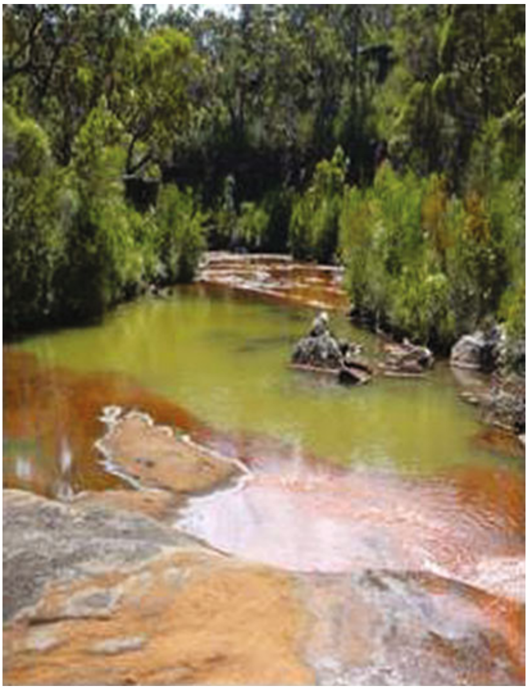


**Fig. 10.32** Discolouration of water and deposition of iron flocs associated with sub-surface flow reporting back to the surface after dissolving marcasite and/or siderite from fresh upstream fracture networks in the bed of the watercourse

beyond the angle of draw being attributed to valley closure. Two notable benchmark cases are the Stanwell Park viaduct in NSW, Australia, which required extensive repair (Hilleard 1988) and the Ryerson State Park Dam in Pennsylvania, USA, which had to be reconstructed (Hebblewhite and Gray 2014). A range of preventative and mitigation measures are available as noted in Sect. 10.4.8.

#### 10.4.7.6 Far-Field Horizontal Movements

Although horizontal displacements have been measured more than 2 km away from mining operations, precise strain measurements reported by Byrnes (1999), field observations and numerical modelling such as that by Pells (2011) indicate that extremely low strains are associated with these movements. Nevertheless, it is plausible that horizontal shear associated with these movements may enhance horizontal conductivity, especially around the perimeter of total extraction workings. Experience suggests that beyond several hundred metres of the mining footprint, induced tensile strains are very small and inconsequential (Waddington and Kay 2001a). Research into this aspect is ongoing.

### 10.4.8 Mitigation and Remediation

Consistent with risk management principles, there are three options for managing risk associated with subsidence impacts, namely:

- Eliminate
- Mitigate
- Tolerate

Adaptive management and remediation find application to all three options. Adaptive management is concerned with monitoring subsidence effects and impacts and, based on these outcomes, modifying the mining plan as mining proceeds so as to maintain effects, impacts and/or consequences within predicted or designated ranges. This can involve actions such as reducing the extent of mining within a panel; altering mining height; or changing the dimensions of subsequent panels based on early warning signs of deviation from planned outcomes. Remediation refers to the activities associated with partially or fully repairing or rehabilitating impacts. As such, it is a measure for controlling the consequences of an impact.

Elimination of subsidence impacts on a feature is achieved by leaving a protective pillar beneath the feature. Mitigation is concerned with reducing the impacts of subsidence, with six common means being:

1. Limiting ground movement. This may be achieved by:
   (a) One or a combination of increasing the width of pillars between panels, restricting mining height, restricting excavation width and restricting how close mining can approach a feature. Hebblewhite et al. (2000) and Walsh et al. (2014) provide examples of

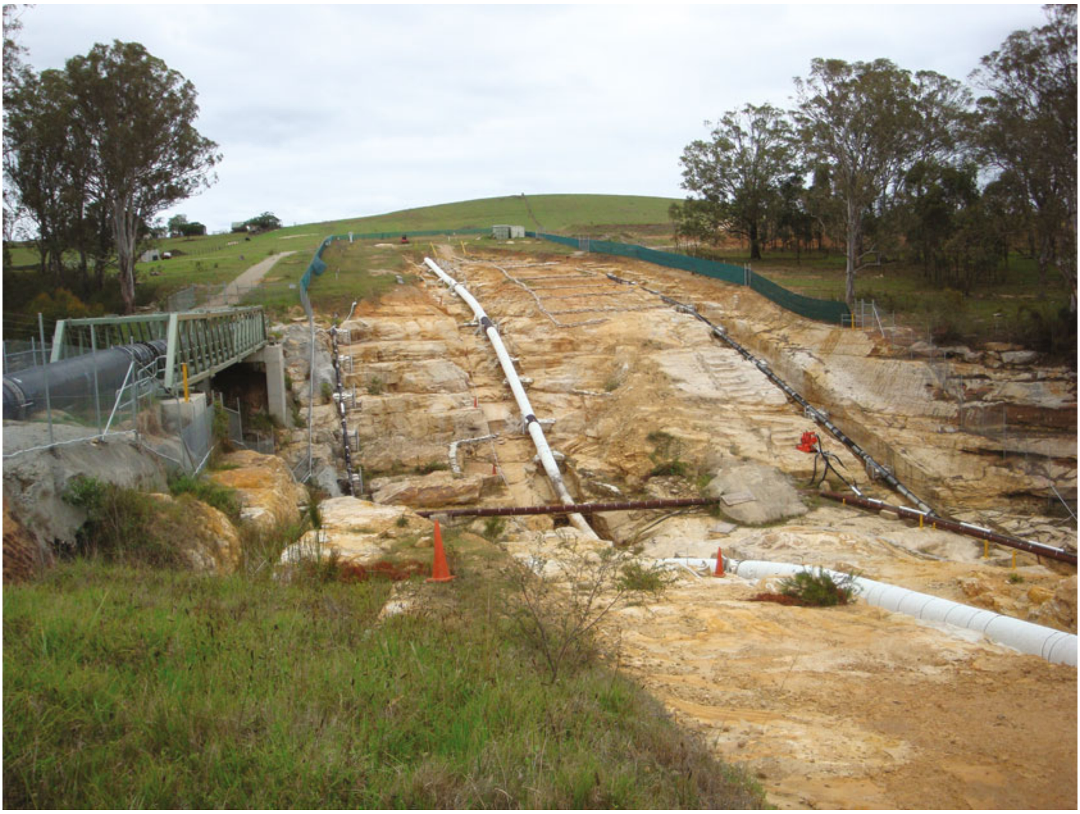


**Fig. 10.33** Decoupling of pressurised gas pipelines and water mains from mine subsidence-induced ground movements, in particular valley closure and upsidence

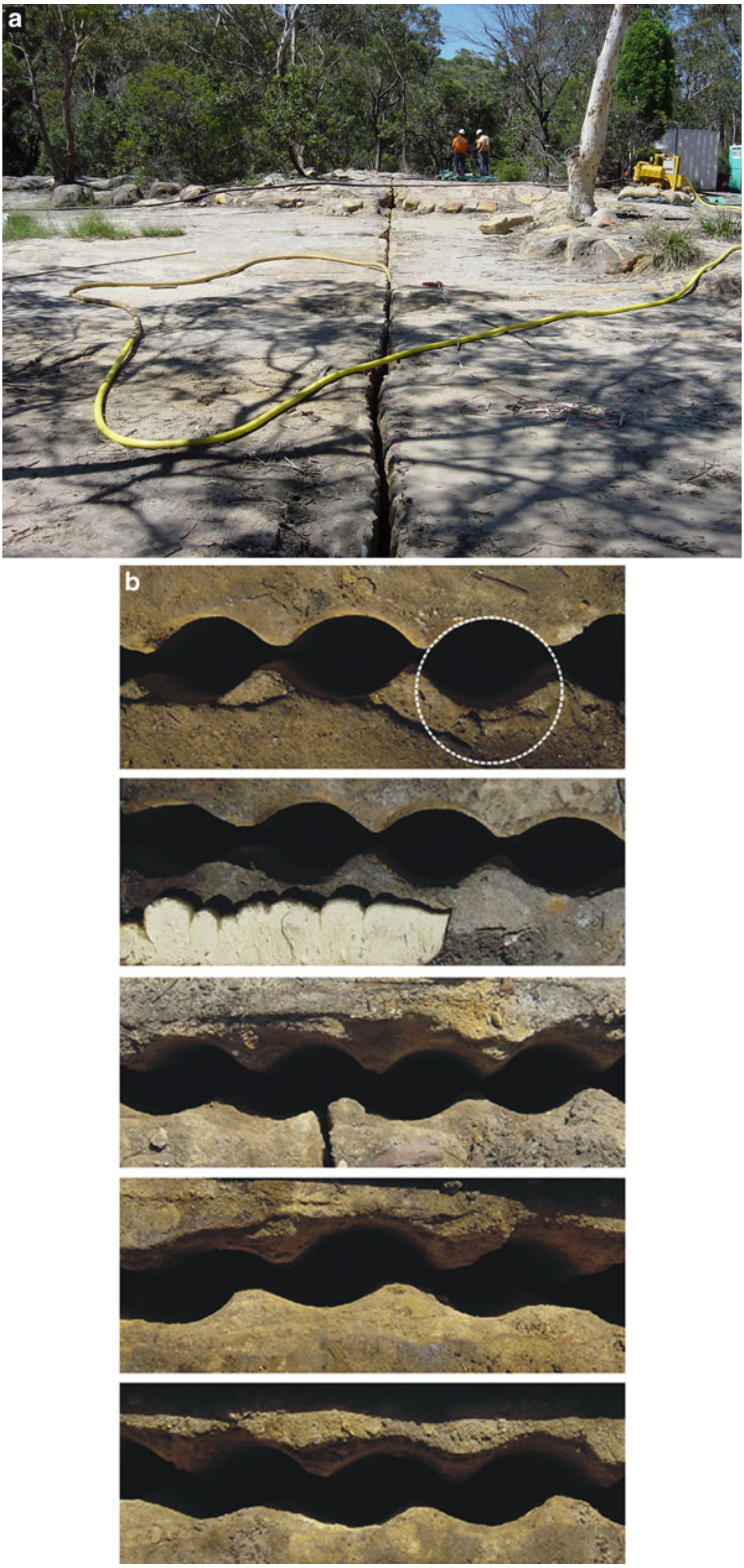
restricting valley closure by limiting the approach distance of longwall mining panels on the basis of real time field monitoring.

(b) **Backfilling** of mine voids, also referred to as **stowing**. Backfilling immediately behind a longwall face has been practiced extensively in coal mines in Europe to reduce surface subsidence, typically by 50–70 %. It is an expensive process that impacts on productivity because mining has to be slowed or curtailed in order to provide sufficient time to place the fill.

(c) Injecting backfill into the parting planes that form in the roof strata in the vicinity of an extraction face as it retreats out of a panel so as to restrict the subsequent closure of the partings. The method finds application in China, where grout is injected at around 100 m above the mining horizon, with the literature reporting 10–60 % reductions in vertical surface displacement. Guo et al. (2005) and Shen et al. (2010, 2014) report on the potential for applying the technique to Australian conditions, which are generally much shallower than those in China.

2. Isolating a feature from ground movement. Techniques include placing bearings beneath structures (such as bridges), uncovering buried structures (such as pipelines), and constructing slots in the ground at strategic locations adjacent to a natural feature to encourage ground movements to concentrate at the slots. Some examples of these types of initiatives are shown in Figs. 10.33 and 10.34. The success of cut-off slots is dependent on a number of factors including selecting the correct locations and directions for the slots,

**Fig. 10.34** Illustration of the use of a slot to mitigate subsidence impacts on a rock bar at Marhnyes Hole on the Georges River in the Southern Coalfield of NSW, Australia. (**a**) A slot created adjacent to a river rock bar by drilling series of closely spaced boreholes in order to mitigate subsidence-induced movements at the rock bar. (**b**) Differential displacement across the slot shown in (**a**), demonstrating the level of protection from strain provided to the rock bar



having access to these sites, and constructing the slots a sufficient time in advance of mining.

3. Rigid ‘floating’ foundations: Placing a structure on a rigid raft foundation that can ‘slip’ on the ground surface enables it to move as an entity, thereby protecting it from curvature and horizontal strain. Rigid raft foundations are often employed in mine subsidence districts in NSW, Australia. However, the structure is still susceptible to tilt, which may be ameliorated proactively by incorporating jacking systems into foundation designs or reactively by injecting expansion agents beneath slabs to re-level them. Rigid foundations can also be retrofitted to structures founded on piers, such as transmission towers (power pylons), in order to prevent differential movement within the structure.
4. Flexible construction: Structures may be designed or retrofitted with a capacity to sustain a degree of differential movement, including valley closure, while remaining safe, serviceable and repairable. An example is shown in Fig. 10.35. Vergara et al. (2011) and Walsh et al. (2014) provide case studies in this regard.
5. Maintenance responses: This involves measures that aim to maintain the physical state and function of a feature, albeit that it may be impacted by subsidence during the mining process. Examples include increasing flow volume in a fractured section of a watercourse in order to maintain surface flow at pre-mining levels; installing support in overhangs and cliff faces prior to undermining; installing tapered sliding rails (switch blades) in railway tracks; and periodically relevelling and realigning man-made structures. Examples of these measures are illustrated in Figs. 10.36 and 10.37.
6. Preservation responses: Objects and features at risk from mine subsidence may be removed on a temporary or permanent basis prior to undermining, or logged and recorded in a visual format for posterity.

Toleration of subsidence impacts does not usually require action be taken to control or remediate the impacts. This practice is common in very deep mines (because subsidence effects are restricted and dissipate gradually over a large area) and at locations that have no significant sub-surface and surface features.

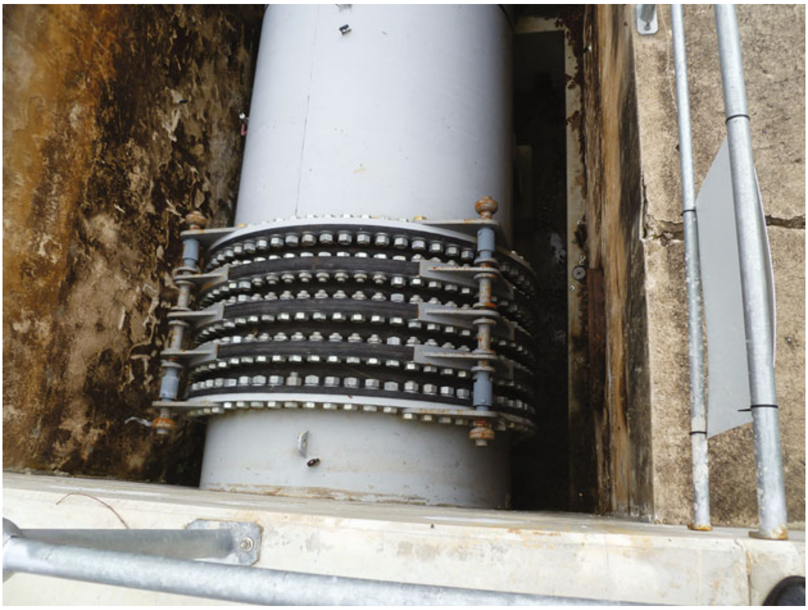


**Fig. 10.35** Rubber bellows to mitigate against valley closure, retrofitted into the abutment end of a 2.44 m diameter wrought iron aqueduct spanning a valley

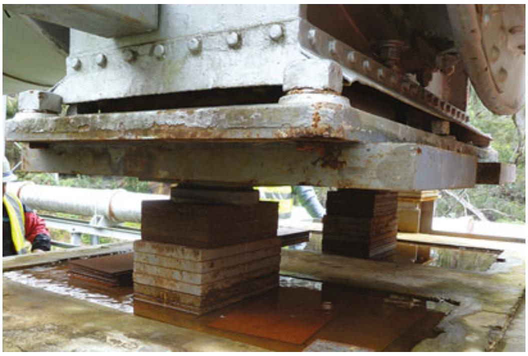


**Fig. 10.36** Jacking and packing of a riveted wrought iron aqueduct spanning a valley to compensate for upsidence during undermining by a longwall panel

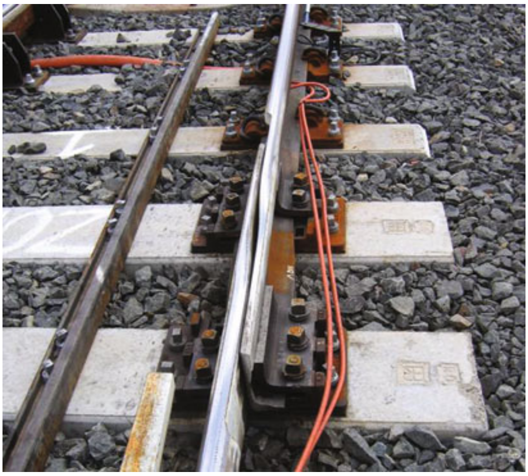


**Fig. 10.37** Tapered switchblades and real time monitoring installed in the main Sydney to Melbourne railway line (Australia) above an active longwall panel in order to manage mining-induced strain in the rails (After Pidgeon et al. 2011)

There is a variety of remediation options available to respond to subsidence impacts. Remediation of the built environment usually involves relevelling and restoring surface finishes, although reconstruction is sometimes undertaken. In the case of natural features, options include backfilling and/or grouting of cracks and fracture networks, stabilisation of slopes, and implementation of drainage and erosion control measures. Fractures may also infill naturally (self-heal) in watercourses that have a moderate to high sediment load; otherwise they may have to be grouted.

Grout can be either cement-based or composed of various polymers or resins (e.g. polyurethane) and can be utilised in one of two ways. The first is the creation of a grout curtain to act as a vertical barrier to the transmission of fluid. The second is shallow pattern grouting to seal the immediate bed of a watercourse. Pattern grouting is not targeted to seal deeper underlying fracture networks.

The degree of success of grouting is dependent on the accessibility of the site, on the type of grouting materials used and on timing. If the site of fracturing is subjected to incremental subsidence by a number of mining panels, several episodes of grouting may be necessary over a number of years. In the interim, mitigation measures are required to sustain surface water flows if the local ecology is not to be impacted.

In the case of watercourses, it is not yet feasible to remediate an entire upsidence fracture network. Hence, remediation efforts to date have focused on sealing the fracture network at strategic locations, such as rock bars. The fracture network can extend some distance laterally under the toe of valleys and be overlain by talus and/or be covered by boulder beds within watercourses. Remediation initiatives may be impeded in these types of settings due to access to inject grout being restricted.

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# Operational Hazards

# 11

**Abstract**

Ground engineering is a risk control measure for a considerable number of operational hazards in underground coal mining. Effective management of these hazards requires consultation and collaboration across a range of disciplines and skill sets. In some cases, the knowledge base concerning the nature of the hazards and their effective control is still evolving and, therefore, it is important that risk management includes provision for monitoring and responding to research and technological developments.

This chapter addresses the operational hazards associated with Windblast; Feather Edging; Top and Bottom Coaling; Inclined Workings; Inrush; Flooding of Mine Workings in the Long Term; Bumps and Pressure Bursts; Gas Outbursts; Mining through Faults and Dykes; Frictional Ignition Involving Rock; Backfilling of Bord and Pillar Workings; Effect of Roof Falls on Pillar Strength; Recovering Roof Falls; Experimental Panels; Alternative Applications for Rock Bolts; and Mining in the Vicinity of Convergence Channels and Paleochannels.

**Keywords**

Airblast • Ashfill • Backfill • Bottom coaling • Bounce • Bump • Buoyancy • Coal burst • Coalbrook colliery • Convergence zones • Dipping workings • Dyke • Dyke drivage • Experimental panel • Fall of ground • Fall recovery • Fault • Fault drivage • Feather edging • Flooded workings • Frictional ignition • Gas outburst • Hydrofracturing • Inclined workings • Inrush • Inundation • Moura mine • Outburst • Overpressure • Paleochannels • Pressure burst • Rockburst • Roof fall recovery • Seismic event • Spiling • Stabilising pillar • Top coaling • Windblast

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## 11.1 Introduction

Ground engineering is a risk control measure for a considerable number of operational hazards in underground coal mining. Effective management of these hazards requires consultation and collaboration across a range of disciplines and skill sets. In some cases, the knowledge base concerning the nature of the hazards and their effective control is still evolving and, therefore, it is important that risk management includes provision for monitoring and responding to research and technological developments.

## 11.2 Windblast

### 11.2.1 Introduction

The phenomenon generally referred to as **windblast** in underground coal mining and as **airblast** in other underground mining sectors is the sudden mass movement of mine atmosphere that has been compressed and displaced by the dynamic failure of support pillars and/or caving of undermined strata. A windblast is characterised by falling strata generating significant overpressures and air velocities. When caving is shallow and slab-like, a negative pressure may also be induced over the top of the falling strata, resulting in a reversal of air flow at high velocity as the event dissipates. This is prone to suck personnel and materials into the goaf (personal experience; Song and Xu 1992; Fowler and Sharma 2000). There is no fixed threshold value of air velocity that qualifies an event as a windblast, although 20 m/s has been nominated in some jurisdictions (for example, MDG-1003 2007; Safe Work Australia 2011).

### 11.2.2 Behaviour Features

Windblasts are usually associated with either a collapse of bord and pillar workings, the subsidence of a large area of standing goaf, or a fall of ground within an open goaf. Some of the most severe windblasts have been associated with the sudden collapse of bord and pillar workings, resulting in multiple loss of life. However, windblasts are most frequently associated with either the initial caving event when commencing to totally extract a panel or falls of ground in the open goaf of partial or total extraction workings. In order for a fall of ground to generate a windblast, the falling material must remain sufficiently intact to act as a piston, the plan area of the fall must be relatively large in order to displace a sufficiently large volume of air, and there must be limited airways available to dissipate the displaced air.

Panel span and the nature of spanning strata are important considerations when assessing the propensity for windblast. The variety of conditions that can be associated with each of these parameters requires that they are assessed on a site specific basis. Local experience can be invaluable in this regard. Windblast events at Newstan Colliery, Australia (see Hebblewhite and Simpson 1997) demonstrate how careful consideration must always be given to whether controls developed to manage one risk may introduce new, and sometimes more serious, risks in other areas.

Visual and audible signs of progressive strata failure preceding a windblast are not always present and even when they are, indications that a windblast is imminent may not be apparent to personnel in the mine. The forces and air volumes involved can cause reversal of flow in main ventilation airways, resulting in impacts extending into zones not normally equipped with controls for managing the associated risks (such as explosion protected electrical equipment). Personal injury can result from organ damage due to overpressure; being blown over; being hit by flying projectiles; being sucked back into the goaf; inundation by displaced water bodies; being exposed to noxious or irrespirable atmosphere; and fire and explosion. The expulsion of flammable gases, the generation of clouds of flammable dust, damage and disruption to ventilation appliances and circuits, and ventilation reversal present an elevated risk of explosion. For example, the Moura No. 4 Colliery explosion in 1986 and the Endeavour Colliery explosion in 1995 in Australia were both immediately preceded by a windblast (described in Appendix 9).

There is a long history of windblast being a feature of pillar extraction, aggravated by the incomplete nature of extraction which can result in caving being impeded by pillar remnants in the goaf. If the mine layout is designed so that these remnants ultimately yield in a controlled manner, they can function as an effective windblast control to regulate the speed of caving. However, when remnant pillars yield in an uncontrolled manner, the situation is similar to that associated with a sudden, domino-like collapse of pillars, resulting in a windblast.

Windblast has come into greater prominence due to the number of mishaps associated with the increased use of partial pillar extraction and narrower total extraction panels in order to restrict surface subsidence or to avoid periodic weighting; the increased use of longwall mining in general, which cannot utilise yielding remnant pillars as a control over caving; and the increased use of longwall mining in strong massive immediate roof settings. These incidents have prompted considerable research, with Fowler and Sharma (2000) presenting the first real time recordings of windblast overpressure-time and velocity-time profiles. These were made in longwall gateroads at Moonee Colliery, Australia. Examples are shown in Fig. 11.1. Similar profiles were recorded subsequently at Newstan Colliery, Australia, in not dissimilar circumstances (Hebblewhite and Simpson 1997). The profiles illustrate the sudden nature of a windblast event, leaving little time for personnel to take evasive action. Flow reversal is not associated with all windblast events.

Longwall panel width at Moonee Colliery was restricted to 100 m in order to limit surface subsidence. Depth of mining ranged from 100 to 160 m and mining height was 3.3 m. The immediate roof comprised 1.5–1.8 m of coal and claystone overlain by around 35 m of pebbly conglomerate strata in a sandy matrix with occasional mudstone lenses. During the 4 year period to 2002, more than 100 significant windblasts occurred at the mine (Fowler and Hebblewhite 2003), with maximum recorded pressure and velocity parameters for these events listed in Table 11.1. The larger windblast events were associated with the lower 10 to 15 m of the immediate conglomerate/sandstone roof caving as a more or less single mass across the full width of the panel and over lengths ranging from 50 to 300 m.

Table 11.1 shows that the peak velocity and overpressure values recorded at Moonee Colliery were 123 m/s and 35 kPa, respectively. Fowler and Sharma (2000) highlighted the very significant hazard presented by windblasts by noting that the lower velocity bound for a Force 12 hurricane on the Beaufort Scale is 33 m/s, or 118 km/h, and that an air velocity of 20 m/s is about the maximum constant velocity against which a human can remain upright. The researchers reported that the velocity threshold for skin laceration is around 15 m/s for a missile weighing 10 g and that the overpressure threshold for eardrum rupture is of the order of 40 kPa, although this may be conservative. The peak overpressure recorded at Moonee Colliery was noted to be capable of generating about 90 kPa when reflected off a ventilation stopping, sufficient to destroy a stopping that is not designed to be explosion resistant.

Microseismic monitoring was introduced at Moonee Colliery during the extraction of Longwall 1 in an attempt to predict the onset of caving with enough warning for personnel to seek refuge from a windblast event. Threshold values were based on the trend, magnitude and frequency of microseismic events (Hayes 2000; Newland et al. 2001). The control proved successful for the first two longwall panels, although there were a number of false alarms and less than 12 s warning on some 16 % of occasions. However, during extraction of the third panel, a windblast occurred with no audible or microseismic warning, resulting in serious injury to a mine worker who was blown some 3 m into a longwall powered support. Mining was halted until a more effective control was devised, resulting in the successful introduction of hydraulic fracturing

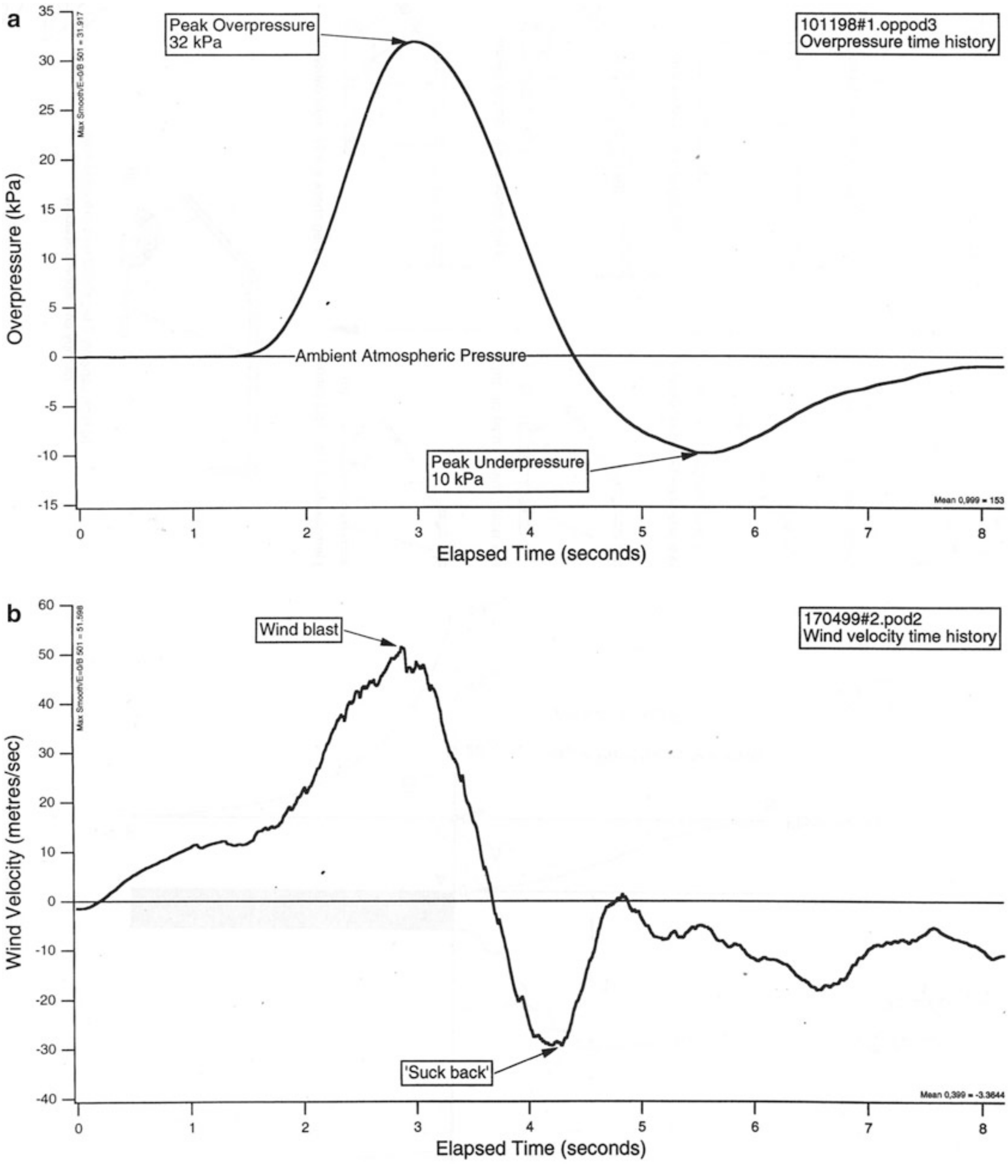

**Fig. 11.1** Examples of overpressure-time and velocity-time recordings made during windblast events. (**a**) Windblast overpressure –v- time recording (Fowler and Sharma 2000). (**b**) Windblast velocity –v- time recording (After Fowler 2001)

**Table 11.1** Maximum recorded overpressure and velocity values associated with windblasts monitored at Moonee Colliery (After Fowler 2001)

| Parameter | Maximum recorded value |
|---|---|
| Peak windblast velocity | 123 m/s = 443 km/h |
| Maximum rate of rise of windblast velocity | 182 m/s$^2$ |
| Peak dynamic pressure | 12 kPa |
| Peak overpressure | 35 kPa |
| Maximum rate of rise of overpressure | 34 kPa/s |

(that is, hydrofracturing) to induce caving at designated times.

The in situ stress field characteristics, the orientation of mine workings relative to this field, the mining-induced stress field and the engineering properties of the rock mass have a significant influence on the likelihood of hydrofracturing proving effective in inducing caving (Jeffrey and Mills 2000; Mills et al. 2001). The success of hydrofracturing for preconditioning strata to cave when undermined is highly dependent on the magnitude and direction of the in situ stress field, as this determines induced fracture orientation. Hydrofracturing has proven more successful in inducing caving after extraction. Mills et al. (2000) attribute this to the trajectory of the minimum principal stress being modified by extraction such that it always projects out over the goaf, thus causing the hydrofractures to also develop in that direction. These features are illustrated in Fig. 11.2.

The hydraulic fracturing operations at Moonee Colliery basically entailed stopping after 20 m of mining, if there had not been any caving, and drilling a hole into the immediate roof between the longwall powered supports at points one-third and two-thirds along the face. These were connected to hoses and mining resumed for some predetermined distance, typically of the order of 60 m. The face was then evacuated while the upper sections of the holes were pressurised to 5–10 MPa to initiate fracturing. Pressure was then reduced to around 1.5 MPa, causing the fracture to grow and, ultimately, to become self-propagating under the weight of the resulting cantilevered strata. Goaf falls usually occurred within 15 min to 2 h of commencing injection, although on occasions, the goaf did not cave until after the resumption of mining. The intervention had the effect of reducing the maximum goaf fall area from 31,560 to 12,600 m$^2$, resulting in a significant reduction in windblast intensity (Hebblewhite 2003).

### 11.2.3 Risk Management of Windblasts

A Windblast Management Plan founded on a risk assessment process provides a tool for managing the risk of windblast. Factors that should be considered when developing systems, standards and procedures to support this plan, presented in an order generally consistent with the hierarchy of risk management controls of eliminate, mitigate or tolerate, include:

- Mine Design
  - Panel span: Ideally, panels should be designed to be either sufficiently wide to induce caving very soon after the start of

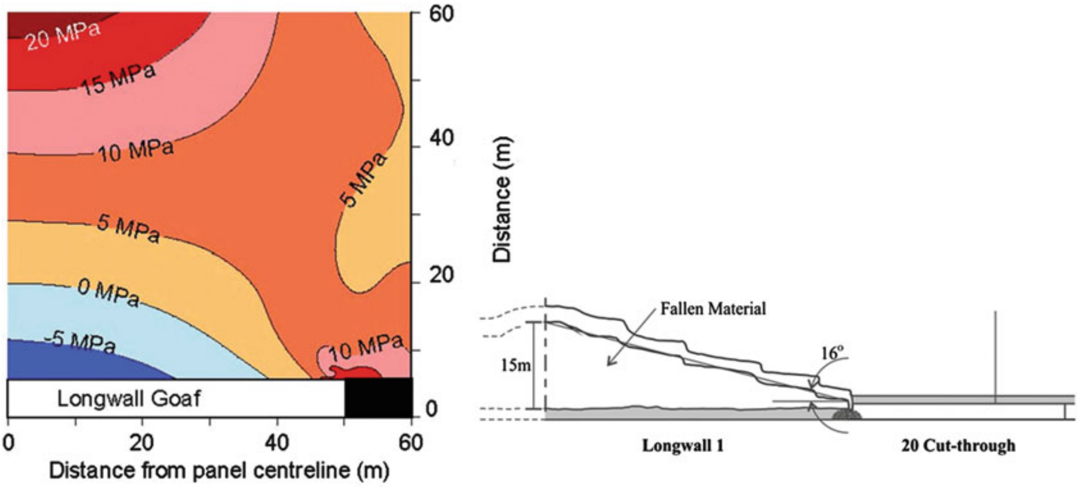

**Fig. 11.2** Correlation between trajectory of minimum principal stress and a typical hydrofracturing-induced caving plane (Adapted from Mills et al. 2000)

extraction and regularly thereafter, or else sufficiently narrow so that goaf falls do not occur or are very small in area. Increasing panel width, however, can introduce new risks, such as periodic weighting, while the risk of windblast associated with the initiation of caving in the panel may not be eliminated. In pillar extraction operations, select pillars may be left in situ to reduce the span of the immediate roof. This approach requires careful geotechnical analysis and risk assessment to quantify any new or elevated risks associated with it. This includes increased abutment stress on the extraction line and an uncontrolled collapse of the in situ pillars, with the later generating a windblast and possibly resulting in pillar failure extending outbye of the extraction line.

  - Parting thickness: Windblasts may be eliminated or their impacts significantly mitigated in total extraction workings by specifying a minimum parting thickness between the seam horizon and the strata that is prone to hang-up. The intention is that the caved parting acts as a cushion to retard the rate of caving of bridging strata and as an obstruction in flow paths between the goaf and the mine workings, with both reducing the overpressure and velocity of the air displaced into the workplace. The required thickness of parting is a function of its bulking properties and the mining height. As noted in Sect. 3.3.2, maintaining a parting thickness of eight to ten times mining height has proven effective when undertaking total extraction beneath dolerite sills in South Africa. Hebblewhite and Simpson (1997) reported that at Newstan Colliery, Australia, windblast became a very serious problem when working a 3.3 m high longwall face beneath a massive sandstone channel once the thickness of the friable parting was less than 7 m.
  - Mining height: Since caving height is proportional to mining height, it follows that a reduction in mining height is a potential control in situations where the parting thickness is insufficient to produce an adequate height of caving. This has proven successful at one Australian longwall operation (Hookham 2004).
  - Panel orientation: The orientation of extraction panels relative to the in situ stress field impacts on the potential to successfully precondition strata by hydrofracturing. Caving development is also influenced by panel orientation relative to joint sets. Extraction faces orientated parallel to joints encourage caving. However, they also present an elevated risk of rib and roof falls, thus requiring a compromised layout. The risks associated with orientating extraction faces within 20° of cleating and jointing usually outweigh the benefits.
  - Pillar dimensions: In theory, by manipulating their width-to-height ratio, pillars can be designed to fail in a gradual, controlled manner so as to avoid displacing a large mass of mine atmosphere suddenly. In practice, this is fraught with risk because the design of yielding pillar layouts is a very complex issue that requires detailed information of panel layout and stiffness and pillar post-failure characteristics and because the consequences of a design failure can be extremely high (including loss of life).
  - Panel entries: Multiple panel entries increase the cross-sectional area available to dissipate a windblast, thus reducing its velocity.
  - Ventilation stoppings: Ventilation stoppings incorporating flaps aid in dissipating a windblast and mitigate against damage to the ventilation circuit.
- Operating Procedures
  - Induced caving: Potential options to induce caving include blasting and hydrofracturing to either precondition strata prior to undermining or to initiate caving after undermining. The preconditioning of dolerite sills in South Africa by blasting has met with mixed success (Latilla and van Wijk

  2003). Caving has been induced with explosives in upper Silesian coalfields of Poland and the Czech Republic (Dvorsky and Konicek 2005; NIOSH 2010), but has not proven successful to date in conglomerate strata in Australia.
  - Holt (1989) reported on the use of hydrofracturing around the world to encourage caving of massive strata and of the first trial undertaken in Australia. This resulted in massive sandstone roof ahead of a longwall face being weakened, leading to a reduction in powered support loadings. However, an attempt to pre-fracture a sandstone channel in a nearby mine was unsuccessful (Hookham 2004). As previously discussed, hydrofracturing proved successful in inducing caving above extraction panels at Moonee Colliery in Australia (Mills et al. 2000, 2001).
  - Microseismic monitoring: This has the benefit of being the only system that can monitor the full region from which a windblast can emanate. However, while microseismic monitoring has been reported to provide adequate warning in a number of instances (e.g. Hookham 2004), it has failed to provide warning of at least one serious incident in Australia and is yet to be proven failsafe. A number of windblast events may need to occur in order for the technology to be calibrated to local geological conditions and failure mechanisms.
  - Support monitoring: Because caving can emanate from well back into the goaf and involve only a relatively small weight of rock, face support systems can be insensitive to impending goaf falls. Nevertheless, the monitoring of support behaviour, especially that of timber props and hydraulic leg pressures on mobile roof supports (MRS) and longwall powered supports, has been known to give warning of impending caving and, therefore, should be considered for inclusion in a Windblast Management Plan.
  - Line of fire: A range of procedures are available to reduce exposure to line of fire in the event of a windblast. These include locating equipment and concentrating activities in roadways that are not in the direct line of fire (such as in cut-throughs in a longwall panel); removing all unnecessary plant and equipment from the face area; tying down all mobile and portable items; minimising the number of personnel at the face, especially leading up to a likely caving situation; linking all exposed personnel to lanyards; maintaining high standards of housekeeping; and providing safe places of refuge.
  - Isolation of electric power: In the event of a windblast, it is critical that electrical power is isolated immediately to avoid persons being caught in moving equipment and to remove ignition sources. Paddle switches activated by the windblast and gas monitoring systems find application for this purpose.
  - Explosion protection and intrinsic safety: Careful consideration needs to be given to the implications of ventilation reversal when zoning mine workings on the basis of exposure to risk associated with flammable atmospheres and in selecting the type and location of electrical equipment permitted within each zone category.
- Personal protective equipment (PPE). This may need to be more extensive and robust than standard PPE, comprising for example, full face helmets, leather apparel, and knee and elbow guards.

*MDG 1003: Windblast Guideline* (MDG-1003 2007) provides further guidance on managing the risk of windblast. When developing and monitoring the effectiveness of a Windblast Management Plan, it should be kept in mind that:

- Initiation of the first caving event in any panel may be delayed and, hence, present a higher risk of windblast.
- Due to extraction of the first panel in a series disturbing the regional stress field and the hydrogeological regime, the caving behaviour

of subsequent panels may be substantially different. Caving may occur more frequently or less frequently in subsequent panels.

- A void with potential to generate a windblast or inrush can exist beneath massive bridging strata some distance up into the caved or fractured zones.

## 11.3 Feather Edging

**Feather edging** is a mining-induced stress phenomenon observed in strong, brittle strata and is characterised by the roof falling as a thin wafer of rock, tapering back to almost a razor sharp edge in many cases. It is associated with strong brittle strata, such as some sandstones and conglomerates, and tends to be most pronounced in the vicinity of goaf corners, where the caving angle is flattest. While the goaf can display signs of impending caving, a feather edge fall may develop without warning, causing timber breaker line props to 'kick-out' or topple and allowing the fall to extend up to 15 m or more into the working place, as illustrated by Fig. 11.3. Fracture planes often ignore pre-existing planes of structural weakness and can propagate through high strength conglomerate pebbles rather than within the matrix (Galvin et al. 1991).

Studies by Shepherd and Singh (1998) of feather edging under strong sandstone roof in a pillar extraction panel at one operation concluded that three elements were associated with the phenomenon, namely a goaf that had been standing for an extended period of time; gradual sagging of the immediate roof over a saucer-shaped area; and elevated lateral compressive roof stresses close to the edges of goaf abutment pillars. The typical goaf fall was reported to produce a low angle caved edge of 10–30° (measured from the horizontal) that may reach a height of 3–5 m some distance into the goaf. The cave often developed by stepping between bedding planes. Based on extensive instrumentation, it was considered advisable for rock bolt breaker lines to comprise two rows of 1.8 m long, fully encapsulated bolts, with a 0.3–0.4 m row spacing and an implied density of at least four bolts per row in a 5.5 m wide bord.

## 11.4 Top Coaling and Bottom Coaling

The terms **top coaling** and **bottom coaling** refer, respectively, to the processes of extracting one or more subsequent slices of coal from the roof or floor of existing workings. A generic feature of both top coaling and bottom coaling is that pillar stiffness progressively changes in the extraction area. This, in turn, means that pillars no longer support an equal share of the overburden load, with the load acting on short pillars in the area of secondary extraction exceeding tributary area load. Safety factors in primary workings need to be increased to account for this increased load.

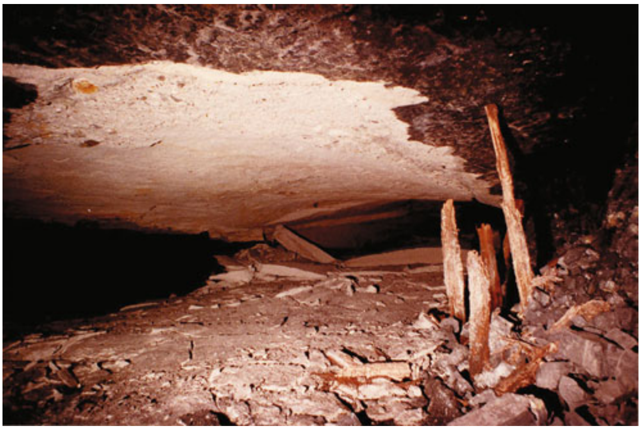

**Fig. 11.3** A feather edge formed in massive sandstone roof and encroaching for over 10 m into the working place from the edge of a pillar extraction goaf

Numerical modelling can assist in quantifying the required increase in pillar strength and, hence, safety factor. Experience indicates that, as a rule of thumb, the Salamon and Munro/UNSW power safety factor of primary workings should be higher than 2.5 and that the final design safety factor for bottom coaling should not be less than 1.6 based on the overall height of extraction. In top coaling operations a final safety factor of 1.4 has been accepted in some countries because top coaling is done on the retreat (whereas bottom coaling can also be done on the advance).

Top coaling has fallen into disuse due to mechanisation and the routine installation of roof support. When it is used, it is important that the brow is supported effectively. Bottom coaling techniques still find extensive application and are particularly attractive in thicker seams because, not only do they require less drivage to achieve a given final design safety factor for the coal pillars, but they also result in a higher overall resource recovery. For example, based on a bord width of 6 m, a depth of mining of 150 m, and a UNSW power safety factor of 1.6, an overall percentage extraction of 26.5 % is achieved from a 6 m thick seam if it is mined to a height of 3 m. If mined to a height of 6 m at the same safety factor, drivage rate reduces to 67 % of the former while overall extraction increases to 37 %.

Bottom coaling requires careful consideration to be given to both rib stability and the practicalities associated with inspecting and, if necessary, re-supporting high roof. Rib spall is likely to penetrate deeper into the pillars, with implications for pillar stability and the size of rib fall events. Support practices and standards during the first mining pass and procedures for monitoring roof stability take on added significance in these circumstances. Additional risk can be associated with bottom coaling during pillar extraction because, firstly, the continuous miner is effectively advancing back into the goaf and, secondly, the ribs of the upper slice may not have been supported. In both first and second workings, it can be advantageous to reduce the width of second and subsequent bottom coaling passes.

## 11.5 Dipping Workings

The principles and analysis presented in this text assume horizontal stratification. Stress distributions and, therefore, ground behaviour are changed if a seam is dipping and workings are driven in the plane of the stratification, as they usually are in moderately dipping seams in underground coal mining. The types of impacts associated with these changes can be conceptualised by simply considering, for the moment, the effect of vertical stress without regard to horizontal stress. Some of the more significant impacts, identified by the points labelled ‘1’ to ‘4’ in Fig. 11.4, are:

- Point ‘1’: The vertical stress distribution over a regular layout of bords and pillars is no longer symmetrical, with the down-dip roof corner of cut-throughs driven on strike being subjected to increasing abutment stress and the up-dip roof corner to stress relaxation. The opposite occurs at the floor horizon.
- Point ‘2’: There is a reduction in the normal component of vertical stress acting on the seam and an increase in the shear component. This shear stress induces down-dip shear displacement of the roof, or **ride**, relative to the floor of the seam, with the seam acting as a shear plane.

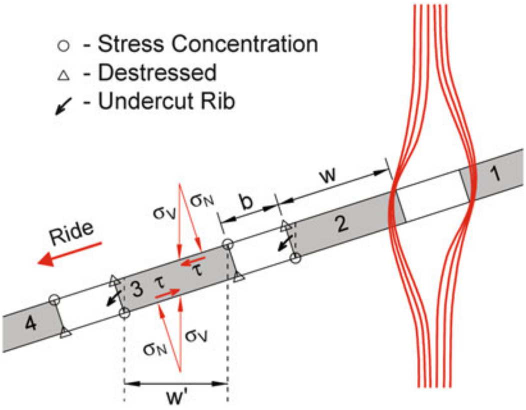

**Fig. 11.4** An illustration of the effect of dip and vertical to horizontal stress ratio on the distribution of major principal stress about a pillar and flanking roadways

- Point ‘3’: A greater portion of the up-dip rib overhangs the bord in cut-throughs driven on strike.
- Point ‘4’: The roof horizon at the face of a heading being driven down-dip is subjected to increased compressive stress, and the roof horizon of a heading being driven up-dip is subjected to stress relaxation. Depending on the primitive stress field, stress relaxation could result in stresses becoming tensile, thus promoting the onset of strata failure in extension.

Experience suggests that coal pillar system stability starts to be adversely affected at dips greater than 5°, although adverse effects can be experienced at lower inclinations in the presence of low shear strength interfaces. The location and extent of impacts are site-specific and are influenced strongly by the stress environment. Numerical modelling is required to develop a proper understanding of stress distributions and their impacts. This is illustrated by Fig. 11.5, which shows the effect of dip and vertical to horizontal stress ratio on the distribution of the major principal stress

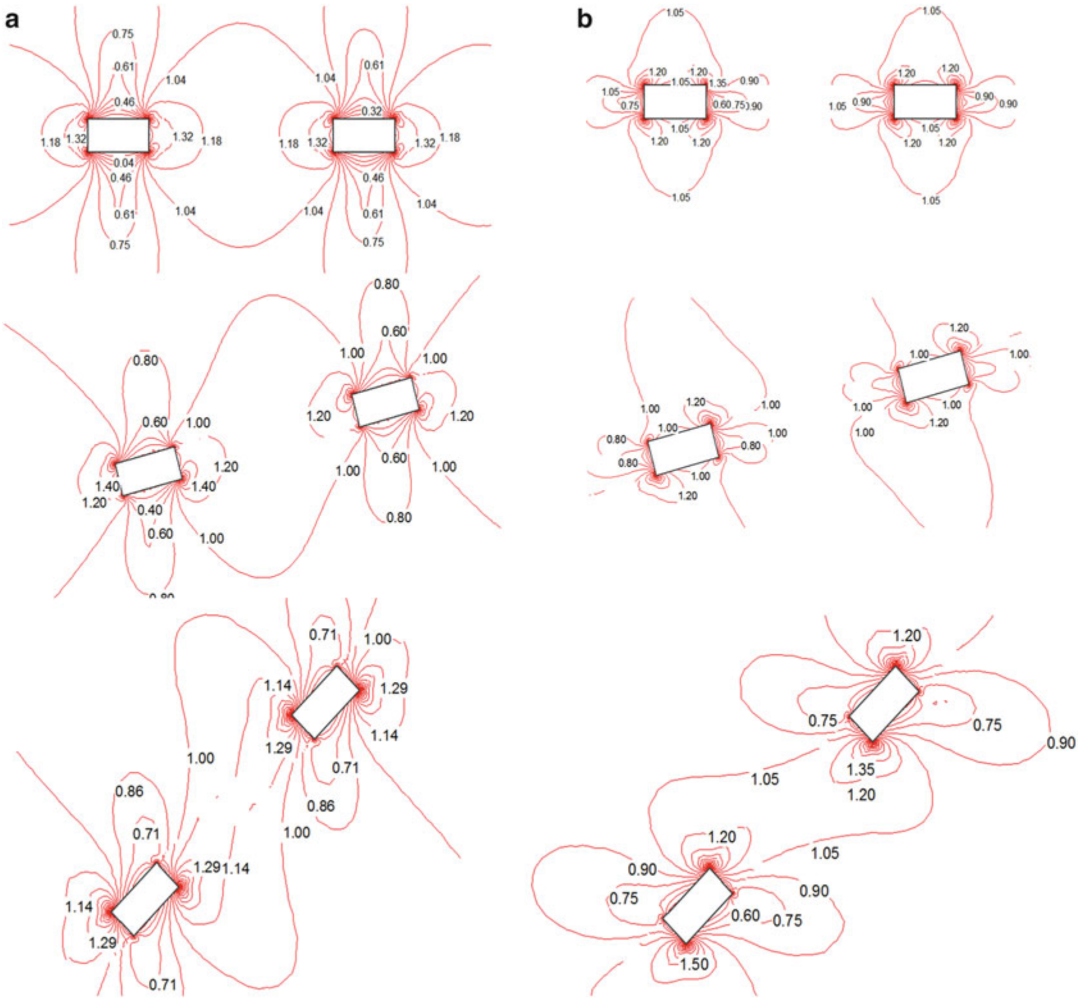

**Fig. 11.5** Stress components and trajectories associated with a dipping seam. (**a**) $\sigma_1/\sigma_z$ for k = 0.5. (**b**) $\sigma_1/\sigma_z$ for k = 2.0

about a pillar and flanking roadways. This example is based on an elastic model in which pillar width was set at 18 m, roadway width at 6 m, and depth of mining at 320 m. The elastic modulus and Poisson's ratio were 10 GPa and 0.3, respectively, and the model boundaries were restrained and located from the mine workings at a distance of 15 times the width of the workings.

The numerical modelling demonstrates how dip induces biased stress distributions within pillars and about roadways and how the locations of the impact points change with vertical to horizontal stress ratio. It gives insight into the state of stress in the immediate roof and floor of the roadways, which can be seen to vary considerably in magnitude. This type of simple elastic modelling can be valuable for giving direction to the type and extent of more detailed field investigations and analytical and numerical modelling that may be required.

Irrespective of the local stress environment, in a dipping seam rib spall from the up-dip sidewalls of excavations can present a serious hazard. This rib line is prone to falling because it is undercut and subjected to stress relaxation at the roof horizon. Ride may also contribute to a deterioration in rib conditions. The presence of low shear strength interfaces at or close to the roof and floor horizons can aggravate ride and increase the detrimental impacts of these interfaces on pillar and rib stability.

In dipping seams, mining conditions are also likely to vary with mining direction. Chen et al. (2000) reported, for example, that generally more roof problems were experienced when mining up-dip than down-dip at Willow Creek Coal Mine in the USA. This behaviour is consistent with theoretical expectations.

When designing mine layouts for seams that dip, the ground engineer needs to consider the maximum operating grade of mobile equipment. Typically, shuttle cars and continuous miners can operate in the dip direction on grades of up to 12° in dry conditions before losing traction. However, they and other mobile equipment are susceptible to sliding into the rib at lower dip angles when working on cross grades. This gives rise to risks of crush injuries to pedestrians and damage to electrical supply cables and has been associated with fatal incidents. Articulated equipment is more prone to rolling over when negotiating corners that are on a cross grade, especially if transporting a load that has a high centre of gravity. The risks are increased when the floor is wet or muddy.

## 11.6 Inrush

### 11.6.1 Definition

Inrush management guidelines developed in response to the findings of the Judicial Inquiry into the Gretley Colliery Inrush in Australia in 1996 (Staunton 1998) describe an **inrush** hazard as involving *the existence of significant quantities of water or other fluid material, any material that flows when wet or flammable or noxious gases, all possibly under pressure, that can swiftly flow or release into or within an underground coal mine* (MDG-1024 2007). The circumstances and causes of inrush are many and varied as evident, for example, by reference to Job (1987a, b), Staunton (1998), Galvin (1998), Forrester et al. (1999), Brady et al. (2003) and RiskGate (2014b).

### 11.6.2 Critical Factors and Considerations

The findings of inquiries into incidents provide a wealth of information as to the causes of inrush events and their avoidance. Nevertheless, some incidents continue to demonstrate that many of the lessons learned from past events have been forgotten or are not fully appreciated. It is not intended to revisit these in this text other than to highlight some critical factors which, based on experience, warrant reinforcing and to briefly touch on selective aspects of ground engineering. Some of these are intertwined with mining engineering, with both ground engineers and mining engineers needing to collaborate in order to effectively manage the risk of inrush. The reader is directed to *Guideline for the Prevention of*

*Inrushes in Mines* (UK Health and Safety Executive 1993), *MDG 1024 - Guideline for Inrush Hazard Management* (MDG-1024 2007) and *RiskGate Bow Tie for Inrush* (RiskGate 2014b) for developing a comprehensive foundation for inrush hazard management.

Critical inrush factors to be alert to are:

- Never, ever, trust a mine plan, including that of your own mine, without thorough investigation as to its accuracy, both in respect of the location and dimensions of the mine workings and their spatial orientation relative to the surface.
- A deeper seam can be a source of inrush if it is under a pressure head.
- Never assume that the recorded location of any surface to seam or inseam borehole is accurate, that the existence of all boreholes is known, or that these holes have been effectively sealed so that they do not constitute a potential source of inrush.
- The pressure acting on a natural or man-made barrier to inrush is not determined by the volume of fluid being retained by the barrier but by the height to which the fluid rises. It is important to appreciate that a column of fluid in a drinking straw generates the same pressure at seam level as a shaft full of fluid to the same height.
- Conventional coal pillar design procedures, in most circumstances, are not appropriate for designing barriers to protect against the inrush of water.
- Geological structures can act as conduits to inrush sources and provide a flow path around barriers and plugs intended to function as controls for this hazard. Conduits can take time to develop, which introduces the risk of personnel becoming conditioned to increasing inflow and not responding to it and/or that the inflow source goes unnoticed or is undetectable because it is in old mine workings.
- Coal seams are natural aquifers and may mask signs of increased water make when workings are approaching a water body. All reports of any water make in the vicinity of potential water bodies should be thoroughly investigated, with mining operations in the area ceasing in the meantime.
- It is foolish in the extreme to dismiss the erosive capacity of water driven by a permanent and substantial water head, such as a river or the sea.
- Sustained water seepage around the perimeter of a bulkhead can lead to an enhanced conduit over time.
- The transition zone between subsided workings and panel abutments as the overlying strata in this zone can be in a permanent state of flexure and, therefore, connected to sources of inrush (see Sects. 5.3.3 and 10.3.2).
- Bedding plane partings induced by mining can have a large fluid storage capacity and provide a lateral transmission network over and under seals and barrier pillars.
- Floor failure in a goaf can instigate a gas inrush from deeper seams and may be aggravated by periodic weighting.
- Drilling into an air pocket to rescue a trapped person presents a risk of the person dying from the bends if the holing-through process is not undertaken within a pressurised environment. Decompression facilities should be on-hand for rescued personnel.
- A water inrush can introduce noxious and flammable gases into a work place due to the release of dissolved gases and the creation of ventilation pressure differentials and ventilation circuits if old workings have been breached.
- Regulatory requirements to drill ahead when within a stipulated distance of a potential source of inrush are premised on the proven location of the boundary of that source of inrush and not on a speculated, assumed, or historically recorded boundary.
- Many guidelines associated with preventing water inrush were developed for specific geotechnical domains and do not have universal application. Examples include tensile strain limits at the beds of water bodies and caving and fracturing heights above total extraction panels (see Sect. 10.3).
- Mine planning needs to give careful consideration to locating mine entries to underground workings such that there is no risk of inrush into the workings from failure of a surface impoundment or a low frequency, high rainfall event (e.g. a 1 in +100 year event).

Plans of old mine workings are notoriously unreliable in respect of direction of mining, mine layout, mining dimensions, and extent of workings. Possible contributing factors include:

- inaccuracies in transferring azimuth bearing underground;
- survey error when turning panels away underground;
- inaccuracies in distance measurements;
- deliberate over-extraction;
- failure to record all workings; and
- failure to update the mine record plan at the time of mine closure.

Unfortunately, modern mines are not immune from some of these hazards, which can range from inadvertently not recording an in-seam borehole or a water collection sump driven by exception, through to over-extraction in a barrier pillar.

Fundamentally, the potential for fluid inflow exists whenever fluid pressure exceeds the minimum principal stress. In underground coal mining, well developed fracture networks may already exist prior to mining or they may be induced by mining. As depth of mining increases, the normal stress acting on fracture networks increases and, therefore, the permeabilities of the fracture networks decrease. However, the development of new fracture networks around the periphery of coal pillars and abutments becomes inevitable as pillar edges yield and crush. Bulkheads can also act as stress raisers and induce bearing capacity failure around their perimeter. It is strongly advisable, therefore, that risk assessment give consideration to the pressure grouting of bulkhead sites and geological structures. Kirkwood and Wu (1995) provide technical considerations for the design and construction of seals to withstand hydraulic pressures in underground mines.

## 11.7 Flooded Workings

The manner in which the flooding of mine workings can impact on pillar load can be conceptualised by considering two extreme but realistic states. The first corresponds to the concept of an hydraulic jack and the second to Archimedes' principal regarding buoyancy.

In the first case, all the surfaces of the mine workings are visualised to be impervious (or sealed), in which case water pressure acting on the roof of the workings functions as a hydraulic jack to unload the pillars. This situation also corresponds to the sudden flooding of a mine in circumstances where there is insufficient time for the water to penetrate the rock mass. Assuming that the pillars are subjected to tributary area load prior to flooding, the load generated on the roof in the area of influence of each pillar is given by Eq. (11.1) for the most general case of a parallelepiped pillar as shown in Fig. 4.4.

$$\begin{aligned} L_{up} &= \rho_f g H_f A_m \\ &= \rho_f g H_f (C_1 C_2 \sin\theta - w_1 w_2 \sin\theta) \end{aligned} \tag{11.1}$$

where

$L_{up}$ = *load acting upwards on roof*
$\rho_f$ = *density of fluid*
$H_f$ = *height of fluid above roof level*
$A_m$ = *area of working extracted; that is, exposed roof area*

Therefore, the effective pillar load, $L_{p\ eff}$, and the effective average pillar stress, $\sigma_{aps\ eff}$, are:

$$\begin{aligned} L_{p\ eff} = \ & \rho_o g H C_1 C_2 \sin\theta \\ & - \rho_f g H_f (C_1 C_2 \sin\theta - w_1 w_2 \sin\theta) \end{aligned} \tag{11.2}$$

*where*

$\rho_o$ = *overall density of overburden*

$$\sigma_{aps\ eff} = \frac{\rho_o g H C_1 C_2 \sin\theta}{w_1 w_2 \sin\theta} - \frac{\rho_f g H_f (C_1 C_2 \sin\theta - w_1 w_2 \sin\theta)}{w_1 w_2 \sin\theta} \tag{11.3}$$

$$= \frac{\rho_o g H - e\rho_f g H_f}{1 - e} = \left(\frac{g}{1 - e}\right)(\rho_o H - e\rho_f H_f) \tag{11.4}$$

With the passage of time, the pores and fractures within the rock mass may become saturated, producing the other extreme where the overburden is fully saturated over the total water head, thus invoking Archimedes' principle. Application of this principle acknowledges that the overburden has buoyancy when it is in a fully saturated state and results in overburden density being reduced by the density of the saturation fluid. Assuming that all pillars were under tributary area load prior to flooding, the effective average pillar stress is now given by Eq. (11.5).

$$\begin{aligned}\sigma_{aps\ eff} &= \frac{\rho_o gH - \rho_f gH_f}{1-e} \\ &= \left(\frac{g}{1-e}\right)\left(\rho_o H - \rho_f H_f\right)\end{aligned} \tag{11.5}$$

If the overburden is fully saturated to the surface, then its effective density is reduced to the order of 1.5 t/m$^3$, resulting in the average pillar working stress being around only 60 % of that for dry workings. This can have a significant positive impact on the stability of old workings, as does the confining effect of water pressure on pillar surfaces. However, the other side of the stability equation must not be overlooked, namely the effect on pillar strength of water under pressure. Water can reduce pillar system strength in a number of ways, including reducing the shear strength of fracture planes and roof and floor interfaces, and accelerating the degradation of any clay rich materials in the pillar and adjacent strata. Pillar collapses associated with the flooding of old bord and pillar iron ore workings in the Lorraine district of France reflect this behaviour. On the other hand, flooding of old bord and pillar coal mine workings beneath the city of Newcastle, Australia, over many decades appears to have had no adverse effect on pillar strength, or else to have been compensated for by the associated pillar unloading.

Careful consideration needs to be given to the possible adverse impacts on pillar system stability of de-watering workings as there is a history of mine workings collapsing soon after being de-watered. This appears to be associated with the strength of the pillar system having deteriorated while in a flooded state and, in some circumstances, with the generation of excessive pore pressures because of rapid drawdown of the water head.

## 11.8 Bumps and Pressure Bursts

### 11.8.1 Definitions

Bumps and pressure bursts are forms of dynamic rock failure involving a sudden release of strain energy stored within the rock mass due to the disturbance of an unstable state of equilibrium (see Sect. 2.6.11 and Fig. 2.28). Instantaneously, strain energy stored in the mass around the region of failure is liberated in the form of kinetic energy. The magnitude of an event depends on the quantity of strain energy that can be transformed into kinetic energy as a result of rock failure or slippage on existing natural and mining induced discontinuities in the rock mass.

There is considerable variation and ambiguity in terminology for dynamic rock failure events in the international underground mining industry in general, although the terminology is clearer in the hard rock sector. This sector makes a distinction between a **strain burst** and a **rockburst**. A stain burst is a comparatively low energy seismic event resulting from the local failure of a small volume of highly stressed rock in the immediate vicinity of an excavation, usually close to the active working face. This type of event can have a magnitude ranging down to $-1$ on the Richter scale and is characterised by the ejection of relatively small pieces of rock. Nevertheless, the ejection velocity can be very high, possibly exceeding 100 m/s (Wagner 2014). A rockburst, on the other hand, is a higher energy event that can range up to magnitude 5 on the Richter scale. Typically, the magnitude of rockbursts encountered in deep tabular gold mines in South Africa ranges from 1 to 3. Most pressure bursts associated with coal mining would classify as strain bursts in the hard rock mining sector.

In the coal mining sector, the terms **pressure bump** and **pressure burst** are most commonly used to describe dynamic energy release events. The commonly accepted difference between the two terms relates to magnitude and, hence, consequence. A pressure bump is a dynamic release of energy within the rock mass that is of sufficient magnitude to generate an audible signal, ground vibration and potential for displacement of loose or fractured material into the mine workings. It is usually associated with either failure of intact rock or failure and displacement along a geological structure. In the USA, a pressure bump is sometimes referred as a **bounce**.

A pressure burst, on the other hand, is a pressure bump that actually results in dynamic rock failure (including coal) in the vicinity of a mining excavation, resulting in high velocity expulsion of the failed material into the excavation. The energy levels and associated velocities are sufficiently high to result in significant damage to, and even destruction of, conventional rock mass support and reinforcement systems.

The distinction between a pressure bump and a pressure burst is better defined in some countries than others. In theory, the term 'seismic event' covers all terminology since the events are all related to the release of energy that causes oscillation of the rock mass to some extent.

In Australia, New Zealand, Germany, Poland and some other countries, the term **outburst** is ascribed to a dynamic event in which high gas pressures play a major role in causing material to be ejected into the workplace (see Sect. 11.9). Outbursts are normally only associated with coal mining because this is where high in situ gas concentrations are more prevalent. However, there are exceptions such as when other strata units close to the mining horizon contain high gas concentrations and when gas is present in evaporate deposits. Caution is required with the use of the term in some countries such as the USA where it is also used to describe purely stress driven dynamic events that would be referred to as pressure bursts in countries such as Australia and Germany.

In this text (unless otherwise quoting from a third party), the terms *bump* and *bounce* are used to refer to a seismic event that can be felt by the human body but does not result in the ejection of material into the workplace. The term *burst* is used to refer to a seismic event that is not associated with high gas pressures and that results in ejection of material into the workplace. On this basis, the origins of a bump could be anywhere within a coal seam or the surrounding strata, while a pressure burst is confined to the coal seam being mined. This approach is consistent with the definition of Bräuner (1994) that:

> *'.....rockbursts [i.e. pressure bursts] in coal mines are violent failures of the coal seam, causing ejection of broken coal and often taking the form of an abrupt movement of the face or sidewall. They may therefore more appropriately be designated as 'coal bursts'. Sometimes they are accompanied by immediate floor heave or, less frequently, by sudden roof fall.'*

Bumps and bursts are usually associated with the presence of strong, stiff strata in the roof. This is consistent with theoretical expectations, as encapsulated in Eq. 2.6. However, it may not be immediately obvious since there are two opposing issues in play, namely that capacity to store energy decreases with increase in elastic modulus but increases with rock strength and the volume of strained rock. Since elastic modulus and strength are related, rocks of high modulus also tend to be stronger and, therefore, result in structures that have a large volume prior to failure. It is the volume of these structures that primarily determines the amount of stored energy, consistent with Eq. 2.6. An example of this behaviour is provided by thick beams of competent rock strata that span great distances before failing and, hence, store large amounts of strain energy.

Distinguishing between a bump and a burst can occasionally be problematic, especially when the ribs are in a supported state prior to the event; when there is no microseismic monitoring system to accurately locate the epicentre of an event; and when there are no witnesses to the event. This can be compounded by the fact that a bump can trigger a burst. There is a history in deeper coal mines of sections of ribline failing in an instant during a seismic event. In situations where the ribs are already supported and the driving forces are not high, the ribs can take on the appearance of having slumped and so it can be difficult, if not impossible, to determine if the ribs have been subjected to a low level burst event or have simply unravelled in response to ground vibrations associated with a bump.

### 11.8.2 Pressure Burst Failure Mechanisms

Pressure bursts in coal mines have been known to occur at depths as low as around 100 m but, consistent with experience in the mining of hard rock tabular deposits, their frequency and magnitude increase with depth. The most common feature of coal seams prone to pressure bursts is their close proximity to strong, stiff strata. Peng (2008) reported that contrary to general belief, a strong roof or floor does not necessarily need to be located directly above or below the coal seam. Rather, the key factor is that the strong roof strata is located within the caving influence zone, which Peng nominated as 8–10 times the mining height.

Pressure burst events and laboratory research suggest that variations in the physical and mechanical properties of coal are not necessarily key factors in determining propensity to bursting (Bräuner 1994; Iannacchione and Zelanko 1995), and that most bituminous coals have a potential to burst (Babcock and Bickel 1984; Iannacchione and Zelanko 1995). Furthermore, bursts can occur in soft coals in a strong roof and floor environment (Peng 2008).

In any situation, four conditions have to be satisfied simultaneously in order for a dynamic (violent) rock failure to occur. The first is self evident and implicit in the other three conditions reported by Salamon and Wagner (1979). These four conditions are:

- The stress environment must be sufficient to result in rock failure.
- A situation must exist which can result in a state of unstable equilibrium. This could be a low friction bedding plane, for example, where the potential exists for the coefficient of friction to drop rapidly from its static to dynamic value once movement is initiated along this plane.
- A change in the loading system. Potential triggers include, for example, a reduction in system strength due to a local change in rock mass material or structural properties, an increase in system stress associated with a local geological structure, or an decrease in confinement due to the formation of one or more excavations.
- A large amount of energy has to be stored in the system. This energy can be generated, for example, by depth of mining, bridging strata or geological structures.

When a system is in a state of stable equilibrium, energy has to be expended to cause further deformation. Conversely, a system that is in a state of unstable equilibrium requires only a very small change to transform it from a stable to an unstable state and to release surplus energy which then overwhelms and accelerates the deformation process.

A common feature of all mechanisms that can lead to an unstable system failure is that in the process of loading the system, the resistance to system deformation reduces at a critical stage in the loading process. The failure process will then be stable or unstable depending on whether the energy required to deform the system further is greater or less than the energy released from the loading system as a result of this drop in resistance. The classical example of stable and unstable equilibrium in practice is that described in Sect. 2.6.11 of a rock specimen subjected to a compression test in the laboratory. In that case, the critical parameters which determine the mode of failure are the post-failure load-deformation characteristics of the rock sample and the unloading stiffness of the loading system.

In general it can be stated that all systems are potentially unstable and have the potential to fail violently when their resistance to deformation drops. Instability can come about as a result of:

- failure of rock in compression (defined by a decrease in strength after deforming the rock beyond its point of maximum resistance to deformation);
- failure of rock in tension (defined as per failure in compression);
- shear failure of rock (also defined as per failure in compression);

- sliding of rock blocks on existing surfaces (defined by a decrease in the magnitude of coefficient of friction from its static value to its dynamic value);
- buckling of rock columns;
- failure of support structures.

Two loading situations need to be distinguished, namely:

- Load controlled systems. Typically, these are systems where a structure is subjected to gravity, or deadweight, loading.
- Displacement controlled systems. Typically, these are systems where a structure is loaded as a result of deformation of the system in reaction to a reduction in the structure's resistance to deformation.

Situations can arise in mining where loading systems can suddenly change their characteristics as a result of changing geological conditions. Classical cases include the dissection of an undermined rock beam by a fault, whereby the stiffness of the loading system is changed from that of a rock beam that is clamped at both ends to that of two cantilevered rock beams, each of which is clamped at one end. An even more severe effect can arise when two geological discontinuities intersect to create an unsupported rock wedge, resulting in a displacement controlled system being transformed into a load controlled system.

In terms of energy released as a result of structural failure, a load controlled failure is more critical than a displacement controlled system as the total potential energy of the rock mass that is allowed to fall will be available to cause damage. In contrast, the energy released from a displacement controlled system depends on the stiffness of the loading system, which can vary substantially. Pillar systems are particularly complex in this regard, as discussed in Sects. 4.3, 4.6 and 4.8.2.

Pressure bursts are associated with energy release as a result of either rock mass failure in compression due to high deviator stress or slippage on a geological discontinuity. Pressure bursts due to rock failure lead to a volumetric failure of rock with limited shear wave energy. It is important to note that this type of failure can be brought about by increasing the maximum principal stress, or by decreasing the minimum principal stress, or by a combination of both. In mining, the more common instigator of failure is a decrease in confining stress as a result of rock excavation rather than an increase in stress, with the perimeter of an excavation reverting from a triaxial state of loading to a uniaxial state of loading.

This concept is illustrated for a pressure burst situation by the Mohr-Coulomb diagram shown in Fig. 11.6. Curve (a) shows the state of stability prior to mining for a situation where horizontal confining stress is equal to one half of the vertical stress (i.e. k = 0.5). The effect of confinement being reduced to zero as a result of forming an excavation is shown by curve (b). This instability may develop suddenly if it results in a step change in the frictional resistance of a failure plane, from its static value ($\phi_s$) to its dynamic value ($\phi_d$). Although pressure bursts tend to be associated with highly stressed environments, Fig. 11.6 shows that reducing confinement by a given amount is more conducive to inducing

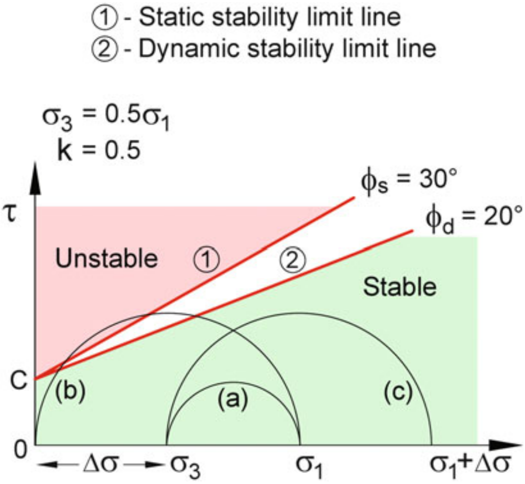

**Fig. 11.6** Mohr-Coulomb plots illustrating the impact of the loss of confinement and change in frictional resistance on the potential for sudden instability, including a pressure burst

instability than increasing loading stress by the same increment.

This example illustrates the sensitivity of the state of equilibrium to the frictional properties of interfaces and the potential for an instability to develop in the sidewalls of a newly formed excavation, even in development workings. It gives insight into the potential for interfaces to play a role in pressure bursts (as reported by Meikle (1965), Iannacchione (1990) and others), as well as into the laboratory research findings of Babcock and Bickel (1984) that pressure bursts can be induced in coal pillars if constraint is lost suddenly. These researchers proposed a modified Mohr-Coulomb failure criterion to describe burst behaviour.

In contrast to dynamic rock mass failure events, pressure bursts on geological discontinuities result in high energy release in the form of shear waves. The failure mechanism can be one of shear movement along natural discontinuities or movement along existing mining-induced fractures. The magnitude of these events depends on the surface area of the discontinuity, the magnitude of the shear displacement and the magnitude of drop in stress. These types of pressure bursts usually have their origin some distance away from the mining horizon.

Pressure bursts can occur in narrow primary development panels. Iannacchione and Zelanko (1995) analysed 172 burst events in USA coal mines between 1936 and 1993 and found that 14 % were in development panels. In these situations, overburden load acting on the pillars is substantially less than tributary area load. Historically, occurrences in narrow panels have been mostly attributable to the presence of a geological disturbance. The disturbance can provide one or two of the four previously noted preconditions required to cause a pressure burst. It can be the source of a high amount of stored energy (that is, high localised stress) and/or the source of instability because its formation has adversely affected the strength of the immediate surrounding rock mass. However, consideration should also be given to the state of stress in a primary development panel and to whether loss of confinement is not, in fact, the primary driver of failure.

### 11.8.3 Seismic Events Associated with Rock Failure

Bumps are associated with strong, stiff strata that may include paleochannels infilled with sandstone and conglomerate. Not all mines that are subject to bumps may also be prone to pressure bursts. However, the occurrence of bumps should prompt an investigation into the propensity for pressure bursts. As a starting point, this may involve applying European auger drilling techniques to assess the state of stress in pillar sides. It is important that these techniques are calibrated to local conditions if a high reliance is to be placed on them as a risk management control measure (see Sect. 11.8.5).

Pressure bursts occur most frequently in underground coal mining in coal pillars close to or at the working face. Pillars can be in a highly stressed state when they are surrounded by wide bords, they are at depth, and/or when they constitute the abutments of total extraction operations. In bord and pillar mining layouts, wide bords can be formed on the retreat by extracting every alternate row of pillars or by stripping one or more sides off the pillars. Abutment stress is elevated when the superincumbent strata contains strong beds which cantilever out over the goaf. These factors increase the risk of a pressure burst close to the mining face in partial and total extraction mining operations conducted at depth beneath competent strata.

Panel pillars close to a pillar extraction goaf are particularly prone to pressure bursts, especially if the face extraction line is not straight and pillars protrude into the goaf. Tailgate chain pillars are also vulnerable to pressure bursts as the longwall face approaches because they are exposed to abutment stress on two fronts and to a high rate of stress change. Longwall faces are not immune from pressure bursts. In nearly all cases, the presence of remnant pillars and goaf edges in nearby adjacent seams significantly increases the risk of a pressure burst when mining under or over these features.

In addition to concluding that pressure bursts can be induced in coal pillars if constraint is lost suddenly, Babcock and Bickel (1984) were of the view that in a mine setting, a strong roof rock can intermittently constrain the coal seam, with frictional sliding or periodic deformation of a soft member between the rock and the coal producing a burst. Field experience confirms that pressure bursts within a coal seam are very often associated with displacement along a roof or floor interface or an intermediate interface, such as at a coal ply or a dirt band. It is not uncommon for a large gap to form between the material involved in a coal burst and the remaining intact portion of the coal seam and for this interface to be coated with a reddish colour dust, to be slickensided, and polished. Coal bursts can be sufficiently violent to push mining equipment into the opposite ribline and to cause significant structural damage.

In situ coal pillar strength tests by Wagner (1974) provide insight into the mechanics of pressure bursts in coal. These tests revealed that microseismic emissions commenced at a pillar stress of about one-half of the pillar strength (Fig. 4.21). Up to the point of peak strength, the events that gave rise to these emissions were typically low energy events that had a high frequency of occurrence. However, in the post failure regime, the number of microseismic events decreased significantly but the incident of high energy events, or large bumps, increased. These larger events were caused by the sudden failure of large volumes of highly stressed coal in the central portion of the coal pillar (Wagner 2014). The removal of the slightest amount of confinement from a coal pillar in the post-failure stage of deformation was sufficient to cause powerful coal bumps. This type of behaviour is observed often in pillar extraction, for example, where the change from high frequency, relatively low energy ‘rock talk’ to massive thumps informs the experienced miner of the state of stress in the workplace.

As discussed in Sect. 4.4.5, Salamon (1995) deduced from analytical modelling of highly stressed coal pillars that a situation could arise where the elastic and yielding zones are suddenly transformed into four zones (by the generation of a crushed zone and a slumped zone), with the potential for this process to be violent and to result in a pressure burst. Pillars with a width-to-height ratio greater than about 7 were postulated to be susceptible to sidewall failure and pressure bursts but would not fail as a whole due solely to these behaviours. This is not inconsistent with the report of Bräuner (1994) that in the Ruhr coal mines of Germany, bursts only occurred in pillars that had a width-to-height ratio in the range of 8–20.

Since mining is usually accountable for the third condition required to initiate pressure burst, namely, a change in the loading system, it is not surprising that the state of unstable equilibria is most often upset at or close to the active mining face. The study of 172 coal bursts in the USA by Iannacchione and Zelanko (1995) found that 78 % of the events occurred during coal production, with 67 % triggered during coal cutting operations. The one documented coal burst in Australia occurred in the rib at the mining face during cutting operations.

This behaviour is related to the fact that the rate of stress change is greater at an active face and that this stress front is moving and continuously exposing new rock that, almost inevitably, will contain an incipient weakness at some point in time. It must be appreciated that stress changes can extend some distance outbye of the face and, therefore, pressure bursts cannot be excluded in these areas, albeit that the risk of an event is significantly lower than at the working face. These factors, in combination with the conditions that need to be satisfied for a pressure burst to occur, result in pressure burst mechanics being a very complex issue.

Once the state of unstable equilibrium has been upset at a specific location, the resulting dynamic energy release can be the trigger for disturbing the state of unstable equilibria at other sites, which may not necessarily be in the same mining horizon. For example, a pressure burst along a pillar ribline may, in turn, instigate floor heave in an adjacent roadway. Bräuner (1994) reported that a number of pressure bursts are known to have happened at the same time as or shortly after shotfiring, even at distances of 30 m or more from the blast area. Ground vibrations due to bumps and pressure bursts can also result in falls of ground remote from the epicentre of the event and damage to mine infrastructure.

### 11.8.4 Seismic Events Associated with Discontinuities

Mining in the vicinity of geological faults or dykes is a well established trigger for a pressure burst. Geological faults constitute clearly defined structural discontinuities, which are often close to limit equilibrium and, therefore, require only very small changes to become unstable. Once movement is reactivated, the angle of friction of the fault plane changes from its static value to a lower dynamic value (see Sect. 2.5.7), with the corresponding drop in coefficient of friction resulting in the release of large amounts of stored energy in the fault area.

This mechanism is described by the excess shear stress concept developed in South Africa by Ryder et al. (1978). It can be explained by considering an isolated total extraction panel. As the panel is extracted, a zone of high compressive stress progressively develops ahead of the face and a stress relieved zone progressively develops behind the face and below the extraction horizon. Therefore, when the face approaches a fault, the fault plane initially comes under the influence of the zone of high compressive stresses. This is usually not critical as the clamping forces on the fault plane increase and, hence, the resistance to slippage increases. Nevertheless, some seismic events may still occur on the feature, as evidenced by the studies at Appin Colliery in Australia, reported in Sect. 3.3.1.

As mining approaches close to the fault plane and as the fault plane is progressively mined through, it falls increasingly within an area of stress relief, thus reducing the clamping forces on the plane. The combination of a reduction in clamping stress and the reversion from a static to a dynamic angle of friction results in a significant reduction in frictional resistance and, thus, in the unclamping of the fault plane that can result in dynamic slippage on this plane. Experience in mining tabular gold deposits in South Africa has shown that the situation deteriorates markedly when the angle between the advancing face and the strike direction of fault becomes less than 20°.

It is very unlikely that underground coal mining can induce stress changes of sufficient magnitude to remobilise fault planes on a regional scale. However, events sufficiently large to cause regional impacts can be associated with localised failures on fault planes. Lasocki and Idziak (1998) reported that in the Upper Silesian Basin in Poland, mining-induced seismicity falls into two distinct classes. Most events occur close to the active mining face and are of low energy. However, there is a second class of events that are higher than $10^6$ J in energy, are more regional in nature, and usually occur at considerable distances from active faces, in correlation with the location of regional discontinuities and zones of weakness. This type of seismicity rarely causes pressure bursts but it often generates damaging effects on the surface. It is believed to be controlled by regional stress build-up on a greater than mine-scale and to be triggered by mining operations.

Large seismic events have occurred on fault planes due to the underground extraction of extensive areas of tabular gold bearing reef in South Africa (Galvin 2002). On one occasion, a magnitude 5.2 earthquake associated with a rockburst at a depth of 1600 m was implicated in the collapse of a residential complex. Seismic events in Australia and the USA have also been related to re-mobilisation of faults (Gale 2004; ANTA 2001; NIOSH 2010), although the regional extent of this mobilisation and the magnitude of the seismic activity were not reported.

### 11.8.5 Risk Management of Pressure Bursts

The nature of pressure bursts makes it effectively impossible to predict their exact location and timing. Control measures come down to mine design in the first instance, followed by monitoring to detect conditions conducive to pressure bursts and then implementation of remedial actions to reduce or eliminate this risk.

The objective of mine design is to avoid creating situations where the four pre-requisite conditions for a pressure burst are present. This cannot always be achieved, especially in deep situations and in the vicinity of geological structures. Furthermore, there is a range of opinions, some of which are diametrically opposed, as to design controls. For example, some advocate that pillars close to the goaf edge should be presplit during pillar extraction in order to move the peak abutment stress further back into the solid, while others are adamant that extraction line pillars should not be presplit. In the case of longwall mining, yield pillars can be effective in mitigating the risk of pressure bursts in gateroads but can increase the risk of pressure bursts on the longwall face at the tailgate end during longwall extraction.

Since pressure bursts are small seismic events, it follows that microseismic monitoring finds application for determining the frequency, location and magnitude of pressure bursts. This information can be valuable in identifying and providing early warning of seismically active areas, or ‘hot spots’, ahead of mining in order to either avoid mining in the area or to implement measures to mitigate the consequences of a pressure burst.

A number of techniques have also been developed to monitor the stress environment about working faces as a basis for implementing avoidance measures. The most extensively applied are based on measuring the quantity of cuttings produced from auger holes drilled into the coal ribs and from the behaviour of the ribs during this drilling process. The higher the state of stress, the greater the hole closure as a result of pressure relief and, therefore, the greater the volume of material removed from the hole. The volume or weight of cuttings and the number of down-the-hole burst events encountered during drilling are correlated to a propensity for bursting. The reliability of these techniques is dependent on a sizable database in order to correlate the measured parameters against bump and burst occurrences. Bräuner (1994) provides more detailed information on these rating systems.

A limitation with all of the described methods for identifying areas susceptible to pressure bursts is that while they are indicators of an approaching critical state, they are not a measure of the actual state of potential instability. They provide no assurance of if or when an event will occur.

If a workplace is identified as having an unacceptable risk of bursting, the options are to eliminate this risk by stopping work or to mitigate it by reducing the level of stress in the vicinity of the workplace and by reducing operator exposure to the consequences of a pressure burst. Measures most often employed at the coal face in an endeavour to transfer stress away from the workplace involve blasting, water infusion, hydrofracturing and large diameter destressing drill holes. All of these control measures are contentious because they may still expose operators engaged in undertaking them to an elevated risk of a pressure burst.

In summary, pressure bursts cannot be predicted but there are measures which can reduce the likelihood of an event in burst prone conditions. Should mining take place in these circumstances, the main focus of risk management then has to be on mitigating the consequences of pressure bursts. Control measures to reduce either or both the likelihood of a pressure burst and the consequences of a pressure burst include:

- Exploration. Determination of the presence and nature of geological structure provides forewarning of possible areas of elevated stress and/or reduced strata resistance to load. Long hole drilling is particularly helpful in this regard. Careful attention should be paid to drilling conditions, such as boggy ground, water loss and high torque requirements, especially if these result in the termination of a hole. There is a range of experience of geological structures going undetected because ground conditions resulted in a hole being terminated due to difficult drilling conditions just before it was about to intersect a feature.
- Stress measurement and stress analysis. Determining the pre-mining state of stress can provide early warning of a potential seismically active area. The timely identification of elevated stress levels and/or a rotation in stress direction may enable mining plans to be

modified accordingly and/or permit pre-emptive measures to be put in place to mitigate the consequences of a pressure burst. Stress analysis prior to mining and ongoing stress measurement during mining provide bases for quantifying mining-induced stress and its impact on total stress and deviator stress, both of which influence the likelihood and consequences of a pressure burst. Consideration also needs to be given to the impact of future mining operations, including secondary extraction and multiseam mining, on the propensity for pressure bursts at a site.

- Mine design. In conditions with the potential to generate pressure bursts, it is advisable to plan the mine workings so as to avoid geological features as much as possible. This includes sandstone channels, joint swarms and shear zones. There is a range of field experience that suggests local geological irregularities, such as faults with small displacements, seam rolls and washouts promote an existing tendency to burst (Bräuner 1994, personal experience).

The design of total extraction panel layouts to encourage caving on a frequent basis is another important control. Regular caving and subsidence limits the volume of strata that can store potential energy. In the case of pillar extraction in potential pressure burst environments, it is important that arrow head pillar extraction sequences (in which a pillar is effectively surrounded by goaf on three sides) are avoided; mining does not take place in interpanel and barrier pillars or between existing goaves; and workings in one seam are not subjected to high abutment stress from workings in an adjacent seam or to high localised stresses associated with remnant pillars in adjacent seams. Details of incidents that illustrate some aspects of these principles are provided by Ramsey (2013) and Barker and McNeely (2014).

As an alternative to caving and subsidence, energy release may be limited by restricting panel width and separating panels by substantial load bearing interpanel pillars, referred to as **stabilising pillars**. There is a long history of this approach proving successful in controlling pressure bursts when extracting deep tabular metalliferous deposits in South Africa (reference, for example, Salamon and Wagner 1979). It also provides the opportunity to locate geological structures within stabilising pillars, provided that pillar design takes account of the effect of these structures on pillar integrity.

The effectiveness of interpanel pillars in controlling pressure bursts in coal mines is reflected by experience in the USA. NIOSH (2010) reported that nearly all of the 17 pressure bursts which affected multiple pillars in the past 25 years occurred in situations where interpanel pillars were either inadequate or absent. The report noted that this finding is consistent with that of Holland (1958), who found that over 80 % of the 163 bursts analysed occurred on ‘pillar points’ (arrow heads – see Sect. 8.4.4) created when interpanel pillars were either subsequently completely extracted or not used in the first place. NIOSH (2010) also reported that 12 of the 17 multi-pillar pressure bursts, including all of those that resulted in fatalities, occurred during pillar extraction and could be attributed to inadequate pillar design. The introduction of longwall mining in place of bord and pillar mining is noted to have been a major contributing factor to reducing pressure bursts in the burst prone Western coalfields of the USA.

Peng (2008) reported that two heading yield pillar gateroad design had been very successful in alleviating chain pillar bumps in longwall operations in Utah, USA. However, he went on to note that it appears pressure burst events increased on the working face, concluding that it is likely that the tailgate yield pillars have transferred the side abutment pressure onto the solid coal face.

Energy release rates are considerably higher when a total extraction face is orientated parallel to an approaching major geological discontinuity than when the face is oriented normal and to one side of such a structure. This is another reason for designing interpanel pillars such that major geological structures are located within them.

Some coal seams within a mining lease can be more prone to pressure bursts than other seams. In these situations, mining operations may be sequenced so that an overlying seam with a lower propensity for pressure bursts is extracted first in order to create stress relief for subsequent operations in other seams.

- Operating practices. Mark (2014) reports that in the case of retreat longwall mining (as practiced in the USA), operational techniques used to reduce the risk of a pressure burst include reducing the depth of the web, reducing the speed of the shearer, uni-directional cutting, and/or avoiding double cuts at the gate ends.
- Destressing. The intention of this control is to destroy the structural integrity of the rock mass so that it cannot store sufficient energy to generate a pressure burst (Peng 2008). This may be achieved on a regional scale (whole of mine) by the pre-extraction of an adjacent seam (see previous discussion point) and on an intermediate scale (panel) by hydrofracturing. Techniques for destressing on a local scale (coal face) include the drilling of large diameter (~100–600 mm) holes into the coal face, shotfiring and water infusion. Care is required with shotfiring as it can also be a trigger for a pressure burst.

  Water infusion is based on the concept that both rock strength and elastic modulus decrease with increasing moisture content. Bräuner (1994) reported that on average, when moisture content is increased from 1 to 5 %, the uniaxial compressive strength of coal was reduced by 30–40 % and the elastic modulus by 30–60 %. One of the major disadvantages of water infusion is that it can take several months to be effective.

  The reader is referred to Bräuner (1994), Baltz and Hucke (2008) and Varley and Whyatt (2008) for further information on destressing techniques. In assessing or adopting these techniques, the end-user needs to give careful consideration to the significance of any differences in the geology and mining method. For example, many European practices were developed in multiseam advancing longwall conditions where the stress regime around a mining excavation and rates of mining during both development and secondary extraction can be quite different to an end-user's local circumstances. Since the strengths of coal measure rocks are prone to be time dependent, rates of mining can have a major impact on the capacity of the rock mass to deform and yield under a high stress situation, with low mining rates tending to be more 'forgiving'. Furthermore, mining in small incremental steps results in smaller step changes and, therefore, there is a lower likelihood of upsetting a state of unstable equilibrium.
- Microseismic monitoring. The potential benefits of this approach have already been noted. It also provides essential information for undertaking fundamental research into pressure bursts and has been applied in the mining of tabular mineral deposits in South Africa since the 1970s (reference, for example, Cook et al. 1977; COMRO 1988). Monitoring of accumulated seismic energy release at one Australian colliery has proven useful in providing warning of impending rock falls that generate windblasts. However, the system is not fail safe (see Sect. 11.2).

  In the absence of a microseismic network, analysis of mining experiences associated with bumps and bursts may provide insight into the root mechanism. For example, some roadway directions or geological features may be more prone to bumps and bursts than others.
- Monitoring of the state of stress based on auger drilling. As the stress on an auger hole increases, it causes the hole to close, resulting in the auger producing an increased volume of cuttings. Volume or weight of cuttings per metre drilled, noise, drill rod behaviour during drilling and pressure bursts in the hole during drilling are used as measures to determine a rating for propensity

to a pressure burst. Propensity increases with the closeness of a highly stressed zone to the ribside. The technique is sensitive to drilling technology, drilling technique and possibly rate of mining and needs to be calibrated to local conditions. Further details in relation to Polish and German operations are provided by Neyman et al. (1972) and Bräuner (1994).

- Minimising the number of persons when working in a burst prone situation, such as when extracting a pillar that protrudes into the goaf, driving a development heading through a structured zone, or installing ground support. Since the likelihood of a pressure burst is higher on an active mining face, it is advisable to remove all persons from the face area during cutting. At the completion of cutting, the face may be bumped with the head of the cutting machine in an attempt to induce a pressure burst prior to persons entering the area. None of these are totally effective risk management options since they still expose some personnel, some of the time, to the risk. They do not reduce the likelihood of an incident and the residual consequences may still be unacceptable.
- Engineered barriers when installing support. Since a pressure burst can still occur sometime after cutting and also some distance back from the face, consideration should be given to persons engaged in installing support doing so either remotely or from within the protection of an engineered barrier.
- Yielding support systems. Yielding support systems have found extensive use for a number of decades in seismically active hard rock mines. Roof support systems primarily revolve around the use of rapid yielding hydraulic props. Rib support systems involve tendons that have an anchorage system which is designed to slip under dynamic loading, integrated with a flexible full surface coverage support system that typically comprises wire mesh and steel rope lacing. Experience in South African gold mines indicates that these support systems need to be able to tolerate ground velocities of the order of 2–3 m/s (COMRO 1988).
- Diligent and frequent inspection and observation. As a change in physical conditions is a trigger for a pressure burst, it follows that diligent inspection and observation by all those involved in the mining process may provide warning of conditions that are susceptible to pressure bursts. This control is not confined to visible changes in the fabric and structure of the strata but also to matters such as bogging of drill steels and changes in location, magnitude and frequency (both increasing and deceasing) of bumping. Changes of particular note in regard to the structure and fabric of coal are an increase in brittleness; change in cleat direction, dip and/or density; and ejection (spitting) of small amounts of material from the roof, floor or ribsides.

The current situation in managing the risk of pressure bursts in pillar extraction and high output longwall operations at depth is reflected in the findings of Varley and Whyatt (2008), who stated that while a number of control systems have been proposed and applied, none have achieved sufficient success in deep western USA coal mines to be considered a viable standard practice. Subsequently, there have been three pressure burst related fatalities in two incidents in pillar extraction in the USA and two fatalities in one gateroad development incident in Australia. The reader is referred to a range of publications concerned with ameliorating the risk of pressure bursts, including Kripakov and Kneisley (1992), Bräuner (1994), Peng (2008), Varley and Whyatt (2008), NIOSH (2010), and RiskGate (2014a).

## 11.9 Gas Outbursts

### 11.9.1 Definition

A **gas outburst** is a phenomenon in which a high concentration of gas usually accompanied by coal is expelled from the roof, floor or sides of a coal mining face. Disturbance is confined to the coal seam and occurs when the pressure of the desorbed gas within the seam exceeds the confinement provided by the rock mass, resulting in an inrush or inflow of material as a fluidised bed propelled by the desorbed gas. Material can be expelled violently or it may ‘flow’ into the workings, with the event having a duration of a few seconds up to several minutes. Pulverised coal and dust can fill the roadway for tens of metres back from the face, leaving a pathway near roof level for escaping gas in the case of large events. Usually, gas emission rates are very high initially and then reduce rapidly with time. The amount of coal ejected can vary from nothing up to several thousand tonnes.

### 11.9.2 Behaviour Features

Although it is unusual for gas outbursts to occur at depths of less than about 180 m, outbursts at shallower depths cannot be excluded. They have also been known to occur on a regular basis with face advance, sometimes as close as 10 m apart. Mt. Davy Coal Mine in New Zealand, for example, experienced nine events over a distance of 300 m. Gas outbursts have also occurred when sheared coal lenses have been present in stone mining faces.

Outbursts are associated with coal that:

- is freshly exposed;
- has a high gas content;
- has been affected by or is in close proximity to compressional geological structures, in particular, strike slip faults, thrust faults and shear zones; and
- is under high stress.

The contributing factors can arguably be reduced to two basic parameters, being rapid desorption rate of gas, and low permeability (GeoGas Systems 1998). Coal permeability decreases with increase in stress. It is common for highly stressed, low permeability zones to be associated with geological structures and for the fabric of the coal to be crushed and pulverised to the extent that it takes on the appearance of mylonite. As a structure is approached, a point is reached where the strength of the intermediate coal is overcome. The fracture network so formed provides the opportunity for gas to desorb rapidly from the coal, to the extent that it may convey coal pneumatically into the workplace.

Seam thickness and the extent of the mylonitised zone are significant factors in determining the size of the gas reservoir and, thus, the consequences of an outburst. Outbursts are often associated with thrust faulting because overriding of strata along the fault plane can increase the effective thickness of the coal seam and produce mylonite and because residual tectonic stress may be elevated.

The gases involved in an outburst are predominantly methane ($CH_4$) and carbon dioxide ($CO_2$). Many operations set a threshold level for total gas content ($CH_4 + CO_2$) in order to eliminate the risk of an outburst. This threshold level is variable and depends on the relative composition of $CH_4$ and $CO_2$, because $CO_2$ has a higher rate of desorption than $CH_4$. In Australia, it is common to apply a sliding scale threshold of around 9–9.5 $m^3/t$ of $CH_4$ in the absence of $CO_2$, reducing to around 6–6.5 $m^3/t$ of $CO_2$ in the absence of $CH_4$. The reader is referred to Kusznir and Farmer (1983), Lama and Bodziony (1996), GeoGas Systems (1998) and Harvey (2002) for further information on gas outbursts.

In the absence of proactive risk management, there is little to definitively warn that mining is approaching an outburst prone situation. Gas emissions may increase, there may be noise associated with stress relaxation, the coal may contain slickensides and be mylonitised, and small eruptions may occur at the face. However, none of these might precede an outburst and if they do, they are easily overlooked or misinterpreted, especially if there is no previous experience of gas outbursts.

### 11.9.3 Risk Management of Outbursts

Consistent with risk management principles, the most effective control is to eliminate the threat

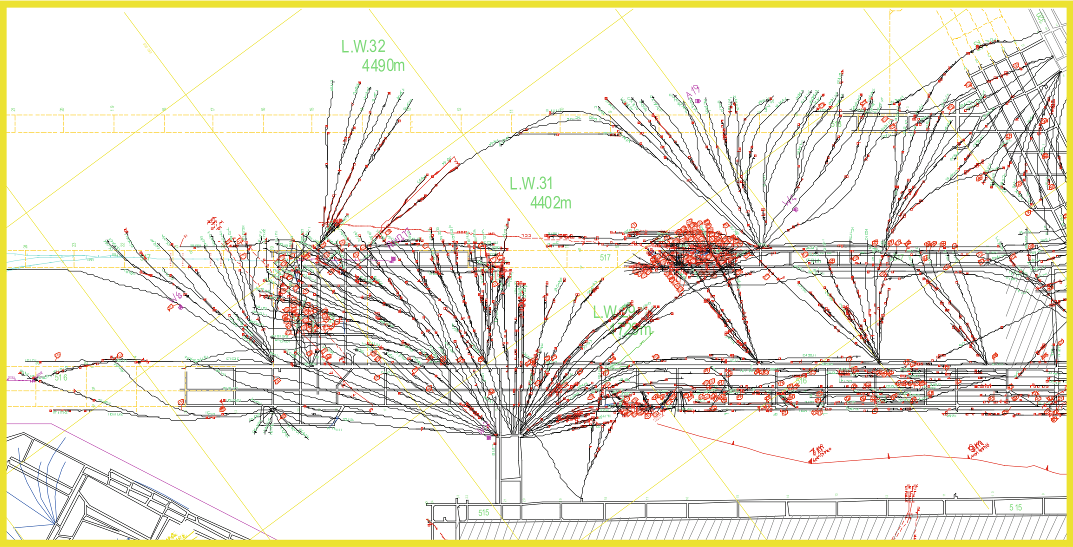

**Fig. 11.7** An example of gas predrainage at a longwall mine in Australia using inseam longholes connected to a vacuum system

by pre-draining the coal seam to below gas threshold levels prior to mining. Inseam longholes drilled either from within the mine workings or from the surface and connected to a vacuum system is a preferred means of predraining gas since it provides the opportunity and time to de-gas a large area well ahead of mining operations. Figure 11.7 shows an example of an in-seam predrainage approach at an Australian longwall mine.

Gas predrainage should have particular regard to geological structures and to the permeability of the coal. Inseam longholes are valuable for detecting the presence and trend of structures. This information, supported by exploration drilling and exposures in mine workings, should be used to project the locations of structures and to target them with predrainage holes. However, the effectiveness of predrainage cannot be taken for granted since some areas can be too impermeable to achieve an adequate reduction in gas content. Therefore, effectiveness needs to be confirmed by testing the gas content of core samples recovered from ahead of each mining face. This works well when it comprises an element of a ‘permit to mine’ system, whereby the mine is broken into small districts with mining of each district only permitted after a set of criteria have been satisfied.

The area of intense drilling to the middle right in Fig. 11.7 is an example of a zone in which considerable difficulty was experienced in reducing gas content to below the threshold limit by predraining. The necessity to go to this extent to reduce gas content is highlighted by the fact that the mine experienced a gas outburst on a longwall face when the panel length was increased by only 30 m into an area that had not been predrained.

As depth of mining increases, it can become increasingly difficult to predrain a coal seam because the high overburden load results in the coal having a low permeability. In these cases, a shallower seam is sometimes extracted first in order to destress the target seam. This overmining method of stress relief is only technically feasible if the risk of gas outburst can be managed effectively in the upper seam. Its use is often constrained by economic considerations. The inability to effectively predrain an area can also be aggravated by mining-induced stress, particularly when the overburden contains very competent beds. In these cases, careful consideration may need to be given to the mine layout, especially panel width-to-depth ratio, W/H, and mining sequence.

A range of less effective measures than gas predrainage have found application and, in some cases, continue to find application in attempting to reduce the risk of gas outburst. These include, in approximate order of increasing effectiveness:

- Modifying continuous miner operations by bumping the face with the continuous miner cutter head prior to each cut-out in an attempt to induce an outburst; limiting the cut-out distance to as little as 0.5 m; limiting the number of personnel at the mining face; and providing these personnel with breathing apparatus and/or an independent supply of fresh air.
- Reverting to shotfiring in an attempt to induce an outburst when no persons are present. This requires the face to be **grunched**, meaning that a horizontal slot is not cut in the coal face to create a second free surface as per standard practice when mining coal by drilling and blasting. No one is permitted to enter the blasted workplace until the elapse of a critical time, typically 30 min.
- Drilling a series of holes into the face and flanks of an advancing roadway to both detect signs of an impending outburst zone and to de-gas the coal ahead of the mining face. By way of example, in one typical operation this comprised eight, 95 mm diameter, 25 m long boreholes, six of which were drilled at two horizons across the face and two of which were drilled at 20° into the flanks. A 5 m overlap was maintained between each series of holes. In these types of situations, it is usual for the volume of cuttings per metre of borehole to be compared against a threshold value to detect if the coal is fluidising as the borehole is being drilled. This methodology has been applied in the German coal mining industry and is described by Paul (1980). Gas flow monitoring from boreholes is also important predictive measure.
- Remote control mining. This still requires persons to work at the face from time to time to install support and advance ventilation.
- Automation, so as to completely remove the need for persons to enter an active mining area.

When undertaking drilling in any situation that has a propensity for a gas outburst, it is important to pay attention to gas make; the nature of the drilling cuttings; and to drilling behaviour. Particular regard should be had to puffs of coal dust blown out of the borehole; very low feed pressures; surges of coal and water; loss of flushing return; blows on the drill rods; drill rods being drawn into the hole; jamming of the drill rods; increased production of rough cuttings; and stress relief noises.

In all gas outburst prone situations, it is critical that high quantities of ventilation are maintained right up to the face and that the atmosphere is monitored with provision to trip all electrical equipment in the event of an outburst. The effective management of risk presented by gas outbursts requires close collaboration between geologists, ground engineers, ventilation officers, mining engineers and mine management. The reader is referred to MDG-1004 (1995), which comprises an outburst mining guideline based on risk management principles. Henderson et al. (2008) and RiskGate (2014c) are other useful points of reference.

## 11.10 Mining Through Faults and Dykes

Geological faults are generally classified in underground mining as:

- Normal fault: A tensional or extensional structure, typically dipping at 70°–90°, characterised by the horizontal distance between corresponding strata remaining constant (when the dip of the fault plane is 90°) or otherwise increasing along the fault plane. Examples of normal faults are shown in Fig. 11.8a, b.
- Reverse fault: A compressional structure, characterised by the over-riding of corresponding strata on the fault plane as illustrated in Fig. 11.8c, d.
- Thrust fault: A reverse fault with a maximum dip in the range of 20°–40°, depending on which definition is adopted.
- Strike-slip fault: A structure in which corresponding strata on the fault plane are displaced along the strike of the fault.

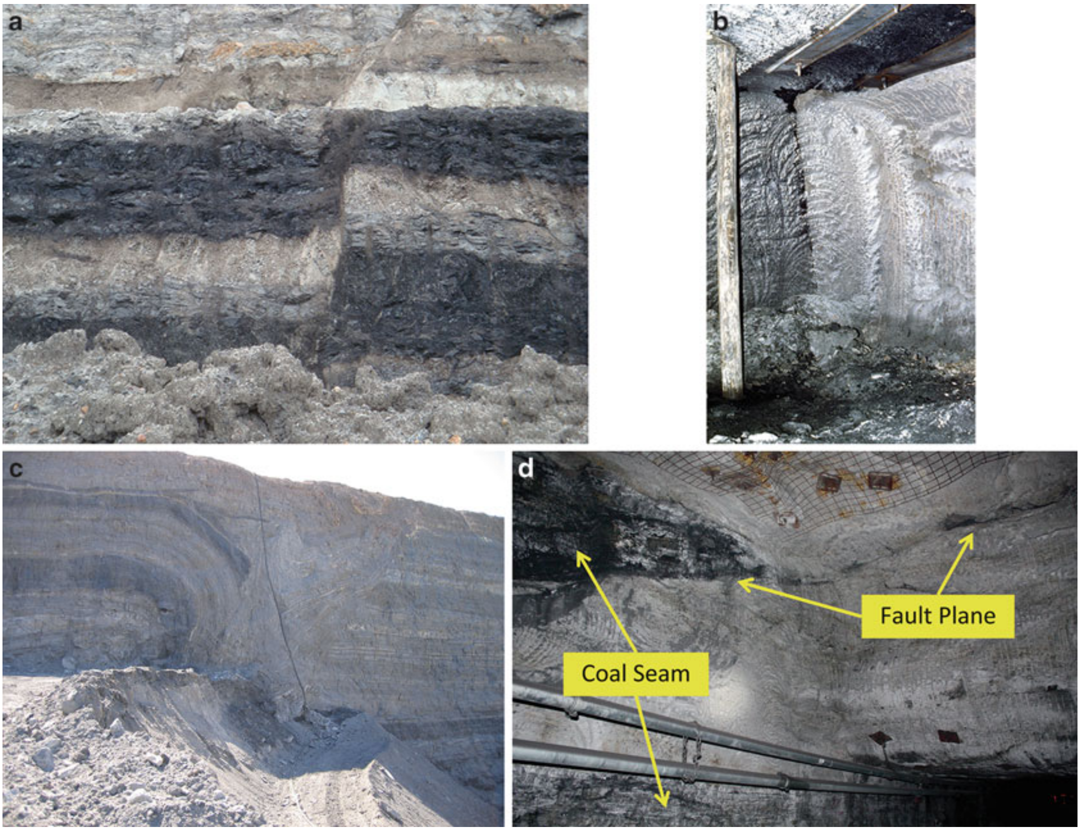

**Fig. 11.8** Surface and underground exposures of normal and reverse faults.(**a**) A normal fault in a surface mine. (**b**) A normal fault in an underground mine. (**c**) A reverse fault in a surface mine. (**d**) A reverse fault in an underground mine

- Bedding plane fault: A fault plane that coincides with a bedding plane. This type of fault may also be classified as a thrust fault.

A geological fault can impact on regional mine stability in three basic ways. Firstly, it can adversely affect the structural integrity of pillars and excavations; secondly, it can reduce or destroy the stiffness of the loading system; and, thirdly, it can cause elevated stress concentrations as mining approaches the fault. Depending on the nature and tectonic history of a fault, stress relief or stress concentration may be associated with it prior to mining. As mining approaches a fault plane, confinement to the intervening strata and the fault plane is progressively reduced while, simultaneously, vertical (or tangential) stresses in this strata become increasingly concentrated between the mining face and the discontinuity as shown in Fig. 11.9. Hence, the deviator stress is continually increasing and frictional resistance on the fault plane is continually reducing.

The impact of these factors on mining conditions can range from one extreme of being undetectable, transitioning through face crew observations such as *'the roof has gone bad'*, *'coal has become soft'*, *'face is spitting'* and *'roof and floor are bumping'*, to the other extreme of pressure bursts and gas outbursts. A pressure burst may be associated with strain bursting of the intervening strata between mining operations and the fault plane or with slippage on the fault plane.

Similar impacts on mine stability and ground behaviour can be associated with dykes. An additional factor to be considered is that coal and rock strata in the vicinity of a dyke tend to be

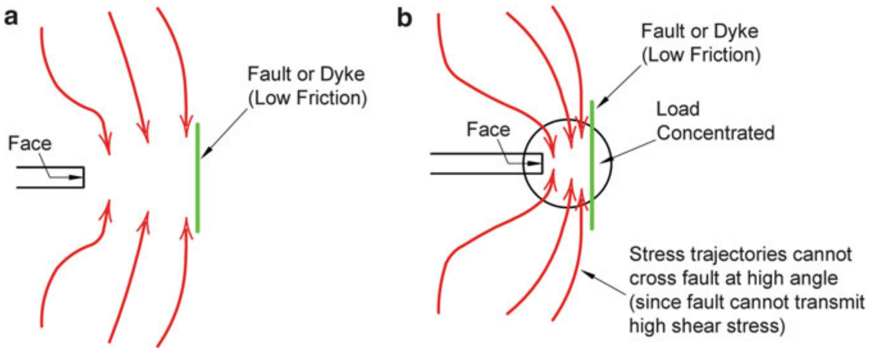

**Fig. 11.9** Effects of a low contact friction geological feature on induced stress distribution ahead of a mining face. (**a**) Induced stress ahead of a mining face. (**b**) Concentration of induced stress as the mining face approaches a low friction geological structure

thermally altered and weakened. The width of these affected areas, which may be different on either side of a dyke, generally increases with the width of the dyke. There is an elevated risk of a fall of ground when mining through these zones.

It is often the case that the presence of a normal fault may not become apparent underground until it is exposed at the coal face, with ground conditions being unchanged right up to that point. Figure 11.8b shows an example of this. Reverse faulting, on the other hand, is usually characterised by gouge on the fault plane and poor roof, face, rib and/or floor conditions for many tens of metres prior to exposing the fault at the coal face. These conditions can be particularly hazardous in the case of thrust faults, especially when their presence is unknown, because the fault plane may track close to the immediate roof and extend back over existing workings for a considerable distance. Consequently, roadway span may increase due to rib spall, effective anchorage of reinforcement systems may be severely compromised, and there is a potential for large wedges to fall out of the roof on the fault plane. In these circumstances, effective ground control may require:

- Cutting down of the immediate roof wedge, as in the case illustrated in Fig. 11.8d.
- The roof to be supported, as opposed to reinforced. This is achieved using stiff cross-supports mounted on high capacity legs to transfer the weight of the roof to the floor, such as illustrated in Fig. 6.53c, d. If the floor conditions are also poor, as is usually the case, it is particularly important that the legs are founded on bearing plates. This type of support system presents a risk to personnel if a leg is accidently displaced by mobile equipment. Therefore, it is advisable to pin cross supports to the roof with bolts, long cables, or saddle brackets. It may also be advisable to angle outer bolts and cables over the rib line in order to transfer the deadweight load acting on a leg to the adjacent pillar should the leg be dislodged or fail.

A number of failures of under-designed bord and pillar workings have been associated with mining through one or more large regional geological structures. The failure may be initiated at the time of first exposing a feature or it may develop at a later time and be bounded by features. Gay et al. (1982) and Kotze (1995) describe a large collapse of chromate pillars in South Africa that occurred without warning between two large fault planes. It was postulated by Gay et al. (1982) that the presence of the fault planes caused the loading system to change from being a stiff displacement controlled system to a soft load controlled system. Another incident involving a sudden collapse of chromate pillars, accompanied by a fatal air blast, was attributed to the coefficient of friction on a fault plane that intersected the workings being reduced as a result of rain water infiltration (Wagner 1991).

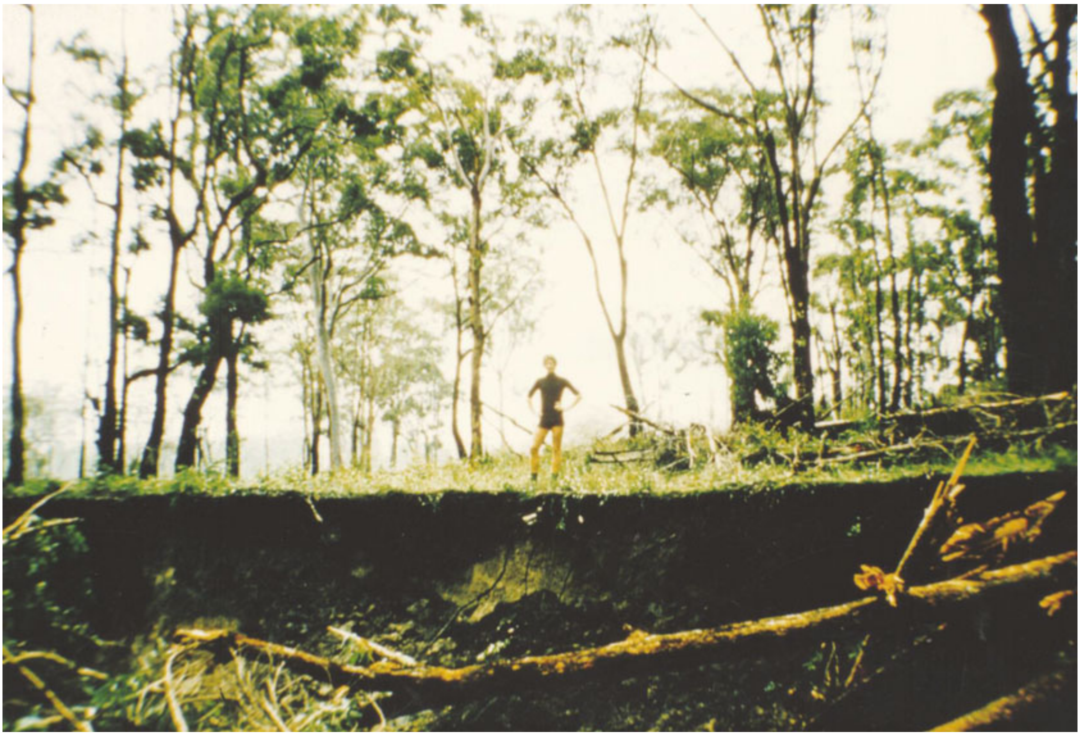

**Fig. 11.10** An example of a pillar system failure that was bounded on one side by a geological fault

Figure 11.10 shows the surface expression of a coal pillar system failure in Australia that was bounded by a fault plane. The presence of a fault or dyke plane can effectively destroy the stiffness of the superincumbent strata and result in a step progression to deadweight loading. Nevertheless, this should be of no concern if pillars are uniform and regular and the pillar system has been properly designed on the basis of tributary area loading. However, the strength and structural integrity of strata in the vicinity of major geological structures can also be adversely affected and this can jeopardise mine stability if not taken into account in pillar system strength determinations.

If a fault or dyke is encountered unexpectedly underground and poor ground conditions are associated with it, drivage may need to be minimised in its vicinity by not forming cut-throughs or by dropping off one or more headings. Otherwise, if it is known prior to mining that poor ground conditions are likely to be associated with a fault or dyke, exposure of the feature should be avoided or minimised in the design process. If this is achieved by incorporating the feature into an interpanel pillar, as is often the case, then, consistent with risk management principles, the design of that pillar should be subjected to change management risk assessment and modified accordingly.

Advances in seismic survey and longhole drilling technologies have improved the detection of faults prior to mining, particularly when the throw of the fault exceeds seam thickness. As floor gradient and effective ground control are particularly important in conveyor and transport roads, it can be advantageous not to mine up to a known geological structure in these roadways until the nature of the disturbance and the methodology for penetrating and effectively supporting it have been determined in other less critical roadways. As a general rule, it is preferable to mine through a fault in a return airway road first because, subject to adequate cross-sectional area for ventilation, unconformities in the profile of these roadways has the least impact on mining operations. This usually occurs as a matter of practice when narrow side brattice ventilation is employed but may require a sequence change when using auxiliary fan ventilation or place changing (cut and flit). The situation is more complicated in longwall mining because the number of headings in a development panel is limited to two or three.

Kelly and Gale (1999) concluded from numerical modelling and microseismic monitoring of fracture development in a longwall panel containing a fault, that the fault played an overwhelming dominant role in fracture development. The fault zone maintained a high level of stress as the longwall approached and was associated with an increased severity of ground failure and displacement. However, in some other cases, faults had no effect on a retreating longwall face (Kelly 2000). Differences in the shear strength properties of the fault plane and in abutment stress loadings may largely account for this behaviour.

A number of impediments and challenges can be associated with longwall mining through a fault. These include:

- Loss of integrity of the immediate roof beam, which then jeopardises tip-to-face control.
- Face spall, resulting in increased tip-to-face distance.
- Soft and weak floor strata, which presents the risk of bearing capacity failure beneath the powered supports, leading to floor heave, uplift of AFC and rotation of the powered supports. Soft floor may also impede clearance and operation of maingate equipment.
- Aligning the longwall face profile with the gateroads.
- Controlling the gradient of the longwall face as it crosses through the fault plane.
- Water make.
- Gas emissions.
- Maintaining ground control at the gate ends.
- The need to cut stone.
- Slow face advance arising from all of the above.

The dip direction of a fault plane relative to the direction of retreat of a longwall panel can have a significant impact on ground stability at the face when the fault is being negotiated. The potential for material to slide off the fault plane into the workplace is higher when the fault is dipping from the face back towards the goaf. In these situations, roof material can unravel to a considerable height well in advance of the face position.

A fundamental control in fault affected ground is for the longwall face to be aligned at least 20° off the strike of the fault. However, other considerations may not always make this possible. In preparing to longwall mine through a fault, it is advisable to:

- Determine the direction and characteristics of the fault over the full width of the longwall block. Mapping of gateroads, seismic survey, radio imaging, in-seam long holes, surface boreholes and correlation to workings in overlying or underlying seams find application.
- Design a ‘flight path’ for optimising roof control and floor grade and minimising cutting of stone when mining between the two coal horizons.
- Precondition the faulted zone.

Depending on circumstances, numerical modelling to gain insight into possible fault behaviour may be warranted. Preconditioning can involve:

- Infusion of the faulted zone with a binding agent injected from the surface through an array of boreholes. This is sometimes referred to as permeation grouting. The primary objectives of grout injection are, firstly, to increase the stiffness of the faulted ground in order to improve stress transfer through it and, secondly, to consolidate the fractured ground prior to exposure by mining. Micro fine cement is used extensively for this purpose. More detailed information is provided in North Goonyella Coal (1999) and AMC Consultants (2006).
- Installation of spiles across the longwall block at the roof level of the planned mining (flight) path through the fault so that the longwall face retreats under the protection of a ‘verandah’.

Hanson et al. (2004) describe the use of 56 mm diameter, 30 mm wall thickness boiler tubes in boreholes up to 130 m long for this purpose at Moranbah North Mine, Australia. The spiles were spaced approximately 1.5 m apart and the holes were pressure grouted with microfine cement.

- Focused support of gate ends in fault affected areas, including installation of long tendons, injection of roof and sides with polyurethane, bulk filling of roof and rib cavities, and the setting of stiff cross supports mounted on steel or large timber legs, timber cribs or cementitious columns. Careful consideration has to be given to the type and location of cross supports to ensure that they do not impede the subsequent passage of the longwall face. Supporting legs have to be out of reach of the shearer or else cuttable, with consideration given to the safety implications if the legs are removed during the passage of the longwall face. These implications relate not only to ground stability but also to hazards such as steel cross supports dropping onto the cutter drum of the shearer while it is rotating. Thought also needs to be given to the implications of face creep, which could be difficult to correct in the presence of poor face or gate end conditions.

It is advisable that a specific Fault Management Plan be developed that covers all aspects of mining through a fault, not just ground control. This plan should be premised on a formal risk assessment process and include a Trigger Action Response Plan and Contingency Plans (see Chap. 12 for more details regarding these risk management aids).

## 11.11 Frictional Ignition Involving Rock

Rock-on-rock and rock-on-metal contact are sources of frictional ignition of flammable atmospheres in coal mines. Rock-on-rock contact is a feature of mining methods that involve caving, while rock-on-metal contact is most often associated with cutter picks striking stone roof or floor or inclusions in the mining face. At the time, rock-on-rock contact was considered a possible but unlikely cause of the Moura No. 4 Coal Mine explosion in Australia in 1986 (Lynn et al. 1987). Subsequent research has not discredited the view of many that rock-on-rock contact was indeed the ignition source. Phillips (1995) reported that five rock-on-rock ignitions occurred in South Africa between 1980 and 1992. There is a long history of frictional ignitions during rock cutting. The now closed Munmorah Colliery in Australia was noted for frictional ignitions when mining to an irregular conglomerate roof in the presence of gas blowers. The mine also had to be sealed and partially flooded in 1989 due to a goaf fire initiated by rock-on-rock contact (Connolly 1990).

Pre-requisites for any fire are fuel, oxygen, and heat, which comprise the so-called **fire triangle**. Contact with rock provides the heat source, with the potential for ignition being a function of the size of the hot spot, its temperature and the period of time that the fuel source is exposed to the hot spot. In the mid 1970s, the British National Coal Board (NCB) issued a standard for classifying the incendivity of rocks in the UK based on their quartz contents as determined by a point counting petrological method (Powell and Billinge 1975). This found widespread international application.

In 1987, the Warden's Inquiry into the Moura No. 4 Coal Mine explosion (Lynn et al. 1987) recommended continued research and experimentation into the phenomenon of frictional ignition for the purpose of establishing a standard whereby all coal related rocks found in the State (Queensland) could be classified according to their degree of incendivity. Initial work undertaken by Ward et al. (2001) recognised that quartz content was not the primary indicator for the frictional ignition potential of Australian rocks, concluding that the main problem with the NCB classification method was probably its neglect of the effect on incendivity potential of rock fragments, feldspar, clays, carbonate and quartz finer than 5 μm. The researchers set about developing an experimental basis for grading rocks from Australian coal mining regions with respect to their frictional incendivity. At around the same time, Golledge et al. (1991) demonstrated the possibility of gas ignition from roof falls in Australian collieries.

The relative propensity for different rock types in Australia to ignite flammable methane atmospheres through frictional contact of rock-on-metal and rock-on-rock was evaluated in the laboratory by Ward et al. (2001) using an instrumented rotating wheel apparatus. A five point incendivity categorisation was developed and the results compared to petrographic composition, mineralogy and related chemical properties. It was concluded that conglomerates, lithic or quartzose sandstones with low clay and carbonate contents, and siliceous cherty tuffs appear to represent the most potentially incendive materials in roof and floor strata, along with siliceous or pyritic impregnations that can occur in the coal seam itself. Temperatures in excess of 1500 °C may be developed within less than 1 s during rock-on-rock contact, with ignition emanating from the contact point or related heat trail rather than incandescent particles (sparks) ejected during the friction process. As a general observation, the rate of heating increases as load is increased up to the point where surface destruction begins to dominate, at which stage the rate of heating declines again.

The management of risk associated with frictional ignitions requires consultation between mining engineers, ventilation officers and ground engineers. Controls of a ground engineering nature which influence the propensity for frictional ignition include:

- the selection of the mining system – first workings, partial extraction with restricted caving, full caving, auger mining;
- the selection of the mining technique – place changing or cut and bolt;
- the selection of the mining horizon – leaving top or bottom coal as a barrier against cutting stone.

Other controls include:

- sharp cutter picks;
- water flushing picks;
- cutting technique - including undercutting coal beneath a stone roof and allowing it to fall under its own weight, and shearing down the face (rather than up the face);
- high ventilation velocity at the coal face;
- gas predrainage, where feasible.

## 11.12 Backfilling of Bord and Pillar Workings

**Backfilling**, or **stowing**, of longwall goaves is used extensively in some European countries to restrict surface subsidence by reducing the height of caving and the void volume of the goaf. It is also used to restrict surface subsidence at some Chinese operations by being injected into parting planes and fracture networks higher up into the roof strata overlying longwall faces (see Sect. 10.4.8). Although backfilling can used for similar purposes in the event of a pillar system becoming unstable, it is primarily used in bord and pillar mining to increase pillar strength by providing active and passive confinement to the pillar sides. Power station flyash has found the most extensive application for this purpose, with cement and aggregates sometimes being added for small scale operations that target the protection of specific surface structures.

Figure 11.11 shows a pure ashfill paste placed in a bord and pillar panel from the surface through a borehole at an Australian coal mine. Backfilling of bord and pillar workings has been undertaken most extensively in South Africa, with around 2 million tonnes per annum of power station flue and clinker ash being placed underground by hydraulic means to enhance mine stability and to dispose of the ash in the late 1970s and 1980s.

Passive confinement to the pillar sides is generated by the pressure that the backfill material exerts on the sides of the pillars, while active confinement is generated in response to lateral pillar dilation. Confinement equates to increasing the width-to-height ratio of the pillars and results in an increase in post-failure stiffness, which means that more energy is required to

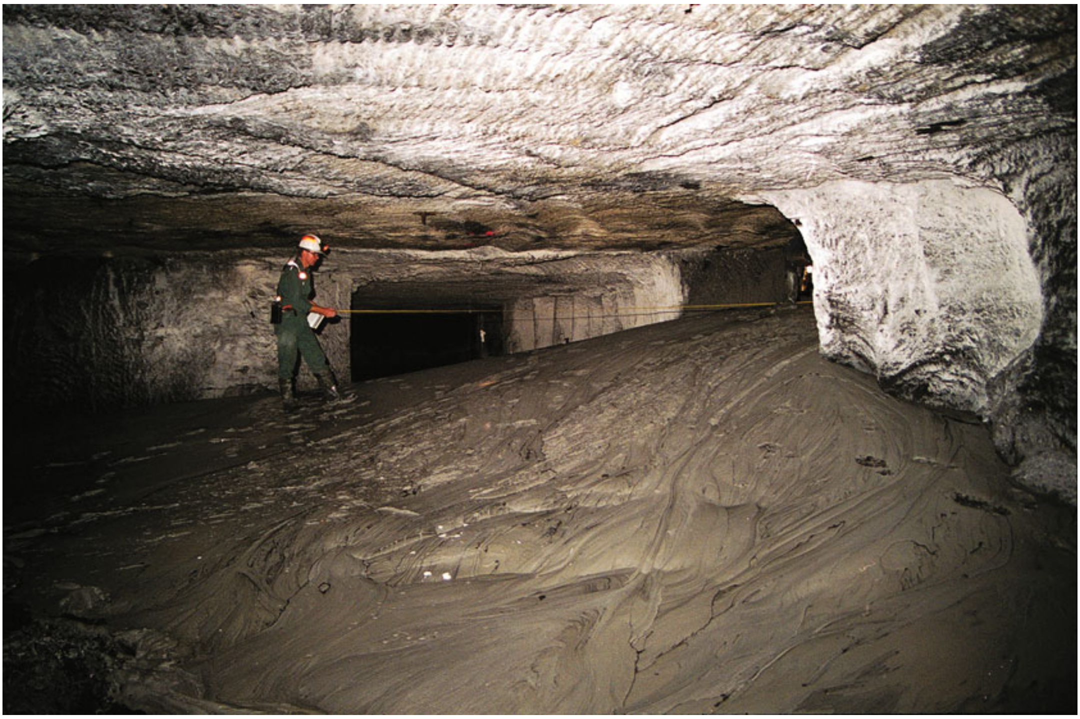

**Fig. 11.11** Bord and pillar workings in the process of being backfilled with flyash paste, Awaba Colliery, Australia

deform the pillar. Hence, the pillar system becomes more stable. Because the lateral expansion of a coal pillar is usually greatest at mid-height, the backfill should extend to at least two-thirds of the pillar height.

The confining pressure generated by backfill in response to lateral pillar dilation is given by Eq. (11.6) (Galvin 1981).

$$\begin{aligned}\sigma_f &= \varepsilon_{lp} E_f = \frac{E_f w \varepsilon_{lp}}{b} = E_f m_\varepsilon \varepsilon_{lp} \\ &= k_f w \varepsilon_{lp}\end{aligned} \tag{11.6}$$

where

$\sigma_f$ = *confining pressure developed by backfill*
$E_f$ = *modulus of deformation of the backfill*
$k_f = E_f/b$ = *stiffness of the backfill*
$\varepsilon_{lp}$ = *lateral pillar strain*
$w$ = *pillar width*
$b$ = *bord width*
$m_\varepsilon = w/b$ = *strain magnification factor*

Equation (11.6) shows that confinement generated in response to pillar dilation increases with increase in the modulus of deformation of the confining material, increase in pillar width, and decrease in bord width. The **strain magnification factor**, $m_\varepsilon$, relates lateral pillar strain, $\varepsilon_{lp}$, to the strain developed in the confining material. Wider pillars benefit more because they are able to tolerate more dilation prior to failure which, in turn, increases the confining pressure generated by the backfill.

There is insufficient field data currently available to categorically quantify the effect of backfill on coal pillar strength. Galvin (1982) conducted compression tests on 150 mm diameter coal specimens of different width-to-height ratios confined by a pozzolanic ashfill in a test configuration that simulated the situation encountered in bord and pillar workings. As shown in Fig. 11.12, it was found that the strength of the confined model coal pillars was

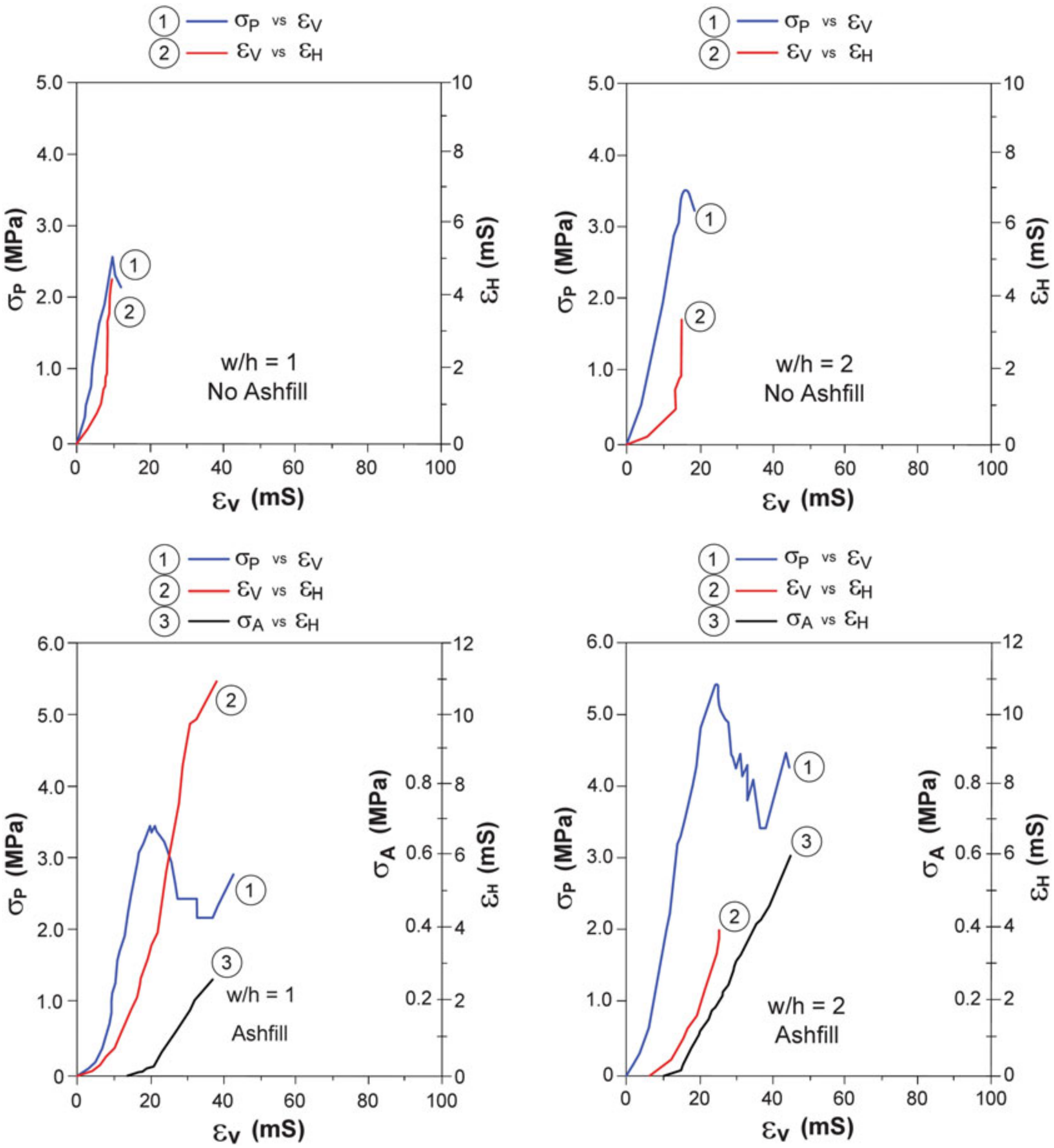

**Fig. 11.12** Comparison between the behaviour of unconfined model coal pillars and model pillars confined by ashfill ($\sigma_P$ = Vertical pillar stress, $\varepsilon_V$ = Vertical pillar strain, $\sigma_A$ = Lateral ash pressure, $\varepsilon_H$ = Lateral pillar strain) (After Galvin 1982)

about 40 % greater than that of unconfined model pillars and that the confinement provided by the ashfill had beneficial effects on the post failure behaviour of the pillars.

An independent estimate of the effect of ashfill on the strength of a 12 m square, 6 m high coal pillar (typical for many South African operations) was made by Wagner (1974) using the failure criterion defined by Eq. 2.13. Wagner assumed that $\sigma_c = 7.2$ MPa (the strength of a unit cube of coal given by Eq. 4.23), $K = 3$ (30° friction angle), bord width = 6 m, fill modulus = 100 MPa (as per Galvin's model pillar tests) and that pillar failure occurred at a lateral strain of $4 \times 10^{-3}$. This produced a strength increase of 2.4 MPa, or 34 %, which was noted to be remarkably similar to the value obtained from Galvin's model pillar experiments. Wagner

(1974) also reported that field observations at Koornfontein Colliery in South Africa showed that bord and pillar workings with nominal safety factors ranging from 0.5 to 0.6 were stabilised successfully by hydraulically placed ashfill, provided that the height of ashfill exceeded 70 % of the pillar height. This would suggest that the stiffness of the ashfill in that specific case resulted in a doubling of pillar strength.

In practice, occasions can arise when it is feasible to backfill only selective sections of the workings. When these workings comprise pillars of varying size, it is not unusual for the backfilling operations to focus on areas containing the smaller or more slender pillars. However, in some cases it can be a more effective risk management strategy to rather utilise the backfill to stiffen the larger or squatter pillars so that they act as regional structural support members across which the superincumbent strata can span. Each situation needs to be assessed on its own merits. It should not be assumed that the optimum benefits will be obtained by backfilling those areas that have the lowest safety factor.

In bord and pillar situations where high to extreme potential consequences are associated with overburden subsidence, it may be wise as an additional risk mitigation measure to fill the workings as close as practical to roof level. If only that portion of mine workings with a potential to impact a surface feature is to be backfilled, design should take account of the abutment load on the perimeter pillars that may arise in the future from a collapse of the surrounding pillars.

## 11.13 Roof Falls

### 11.13.1 Effect on Pillar Strength

Some of the principles applying to the function of backfill find application when considering the effect of roof falls on pillar stability. A roof fall might be thought of as increasing the pillar height and, therefore, causing a reduction in pillar strength. However, some advocate that the fallen material can be considered to function as backfill and to provide confinement to the bottom portion of the pillar to partially compensate for the increase in pillar height.

If the material comprising the roof strata has a notably higher rock mass strength than the coal comprising the pillar, the coal portion of the pillar may still primarily determine the pillar strength. However, if the immediate roof strata is of a similar rock mass strength to the coal, such as for the case illustrated in Fig. 11.13, a roof fall increases the effective height of the pillar. A critical situation arises if the exposed roof strata contains beds that are weaker than the coal, especially if this material extrudes or results in roof falls extending over the top of pillars.

Fallen material has the potential to function as backfill, with active confinement being a function of the shape and height of the muck pile (fallen material) and its bulk density. Figure 11.14 shows a roof fall in a thick coal seam overlain by weak strata. The fallen material has bulked very little and is heaped against the pillar sides and so it is feasible that this material will

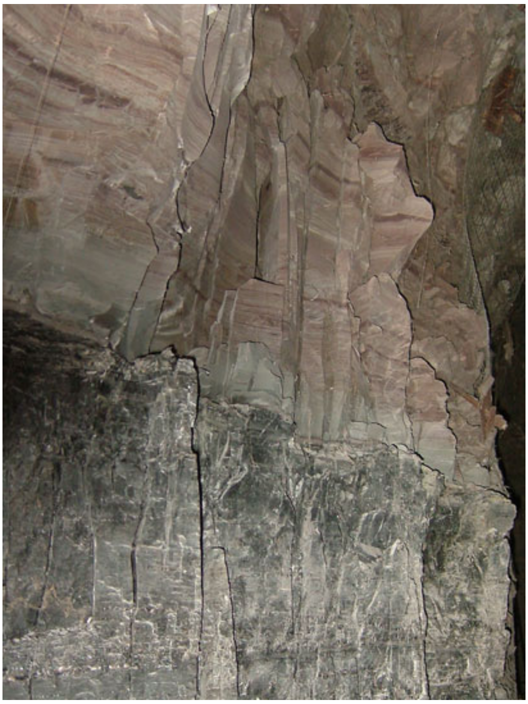

**Fig. 11.13** A fall of roof material which has a similar rock mass strength to the coal seam, thereby resulting in an increase in the effective height of the pillar

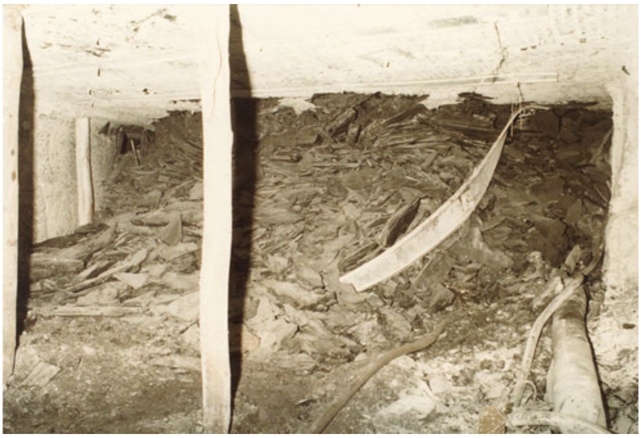

**Fig. 11.14** Roof fall material which is providing a degree of confinement to adjacent coal pillars

provide a degree of confinement to the pillar. However, care is required in applying this concept since it is not only the presence of the confining material that matters but also its height and stiffness. As with backfill to improve pillar stability, it is advisable that the fallen material extends up to at least two-thirds of the effective pillar height if any degree of reliance is to be placed on its confining benefits.

### 11.13.2 Roof Fall Recovery

Roof falls in underground coal mines develop to a point where a stable state is reached because either the fall chokes itself off due to bulking, intercepts a competent bed, or domes out. In many instances, as a result of being subjected to high abutment stress, the strata and the immediate surrounds of the fall site are in an extensively fractured state prior to falling. It is common for falls to occur after this abutment stress has been relieved, thereby removing confinement to the fractured material. During the process of removing material from a fall, operators are exposed to the risk of more material unravelling and falling from height into their workplace. This risk is also present if ground support needs to be installed in the roof and sides of a fall cavity.

In the case of a roof fall in a roadway, all access points to the fall should be barricaded, or fenced off, at a safe distance back from the fall so that persons do not inadvertently enter the danger zone. If the fall has to be prevented from spreading or if a passage way has to be re-established through the fallen area, then the access roads to the fall site need to be re-supported as necessary to permit the lips of the fall to be secured. The lips may be secured temporarily or permanently by setting one or a combination of large legs, timber chocks, cable bolts and substantial cross supports, such as ‘top hats’. It is essential that the lip of the fall is secured at each workplace.

The traditional approach to recovering a roof fall has been to secure the lips of the fall and to then establish a route to the top of the fall, setting short props (‘tommy’ props) off the muck pile along the way. Hand held bolting equipment is then taken to the top of the fall and the roof re-supported with tendons, followed by the sides of the fall cavity. Depending on the nature of the fall, this may require material to be mucked (removed) from the fall in order to provide adequate access and working room. During this process, it is easy to remove too much material, such that the roof of the fall cannot be reached in order to re-support it. Once the fall cavity is re-supported, the fallen material is removed.

Alternatively, in some situations such as where the height of a fall continues to increase as material is mucked out; when it is not feasible or practical to convey the mucked material out of the mine or to stow it underground; or where the primary objective is only to recover equipment buried or trapped by a fall, the fall may be

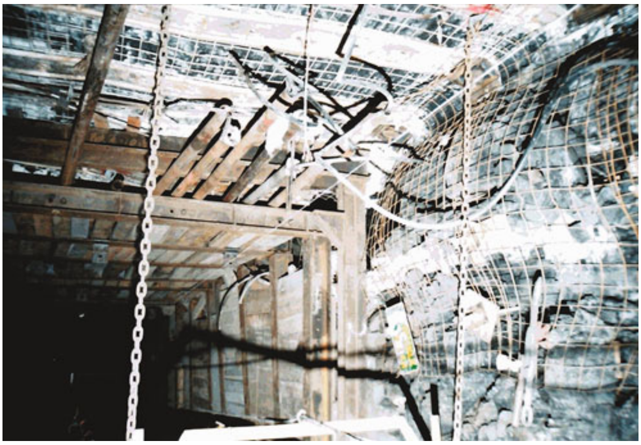

**Fig. 11.15** An example of the use of spiling to restore access through a roof fall

recovered by the **spiling** technique. This involves driving a series of closely spaced bars, or **spiles,** into the muck pile at the working roof horizon so as to form a false roof, or **verandah**, in the muck pile. Steel sets are then set under the spiles and lagged with timber boards as the fallen material is progressively mucked out. Figures 8.32 and 11.15 show examples of spiling. Spiles may comprise pointed railway lines or, as is now common, drill rods fitted with a sacrificial cutting bit and backfilled with concrete. Typically, spiles are 6–12 m long and are angled slightly up into the muck pile at 0.3–0.5 m centres.

The safety and success of spiling is premised strongly on the following precautions:

- Keeping the fall recovery span to a minimum.
- Securely anchoring the initial steel set to solid ground. This initial set, sometimes referred to as the ‘goalpost’, may be purpose designed with an extended base to facilitate its anchorage and a cross guide at roof level to aid in placing the spiles at a uniform elevation.
- Cross bracing subsequent sets.
- Placing spiles at a close spacing.
- Not overdrawing the muck pile prior to placing each steel set.
- Maintaining a sufficient overlap, typically at least 5 m, between each series of spiles.

Application of a risk management approach to the recovery of falls of ground highlights the risks associated with the traditional approach of working off the muck pile to secure the cavity of the roof fall. Hence, this technique has now been discontinued in many mining jurisdictions. However, it may still be required if persons are trapped under a fall of ground and time is of the essence in recovering them. Emergency response management plans need to make provision for this situation. At Tshikondeni Colliery in South Africa, for example, ten rescuers died and seven were injured in 1996 in the process of working on top of a 4–5 m high roof fall to reach an operator trapped in the cab of a roadheader buried by a roof fall at an intersection in a pillar extraction panel. The secondary fall occurred some 3 h after the initial fall and ranged in thickness between 0.6 and 1.2 m.

The development of strata binders, bulk void fillers and cable bolts has facilitated a move to safer methods of recovering roof falls. These are based around utilising a strata binder to stabilise the surrounds of a fall and injecting the muck pile with a bulk filler to form a solid mass, which is then re-mined and supported with long cables anchored in competent upper strata and over the solid/consolidated abutments of the fall. Coutts and Payne (2010) describe a situation where falls were successfully consolidated by injection from the surface prior to re-mining. Pull tests indicated bolt anchorage of some 100 kN (10 t) in the reconsolidated material. As with spiling, this technique minimises the amount of material for which stowage space has to be found underground or which otherwise has to be transported out of the mine. It offers significant advantages over spiling in respect of reduced material handling requirements and demands.

Augering of one or more large holes (1–1.5 m in diameter) provides an alternative approach to fall recovery in some circumstances. It has particular application to re-establishing airways where material has fallen as a large block or where it is not required to support the cavity of the fall. In the later case, material can be recovered from the muck pile at a safe distance until such time as an adequate airway has been re-established over the fall.

In some situations, there is potential to adopt a hard rock mining approach to fall recovery by firstly utilising a remotely operated telescopic arm to spray fibre-reinforced shotcrete on the roof of the fall. Rock bolts are then installed by a remotely operated automatic roof bolting rig as the fallen material is progressively mucked (loaded) out.

In the case of falls of ground on a longwall face, the traditional approach to recovery has revolved around installing steel girders between the powered supports and the coal face to act as forepoles, and to then place some form of packing in the void between these and the roof of the fall. Once again, a risk management approach to the recovery of a longwall face fall largely rules out this methodology today. Instead, it has been replaced in many instances by stabilising the coal face with a strata binder injected from a safe workplace behind the AFC spill plates, injecting the fall and roof cavity with a void filler, and then re-mining through the fallen zone. In some situations, spiling may be used to create a false roof ahead of the face and/or tendons and dowels may be installed in the face area. If this support cannot be installed from behind the protection of the spill plates, temporary support is strongly advisable to protect against the threat of rib and roof falls. Sheets of mesh held in place with air legs and supplemented with powered support face sprags, if fitted (see Sect. 9.3.1), provide one potential form of temporary support to protect against rib falls.

## 11.14 Experimental Panels

There is a history in underground coal mining of pillar collapses arising from the adoption of mining layouts that were first trialled in a so-called ‘experimental panel’. Three factors that have a major bearing on the confidence that can be placed in mining layouts evaluated in this manner are:

- Loading regime: The dimensions of a single experimental panel must be sufficiently large to generate the loads that the pillar system and surrounding strata will experience in routine use.
- Time: Often, layouts trialled in an experimental panel are motivated by a desire to maximise percentage extraction. This implies maximising strata loading, particularly that of the pillar system. Because the strength of rock can decrease over time, the final effects of system loading may not become apparent until well after the completion of the experimental panel.
- Probability of failure: Consistent with maximising extraction, the pillar system in an experimental panel is often designed to a low safety factor. However, there can still be a high probability that a single experimental panel will perform satisfactorily. For example, a UNSW power safety factor of 1.2 corresponds to what is generally agreed to be an unacceptably high 1 in 10 chance of collapse (Table 4.4). Nevertheless, this means that there still is a 90 % chance that an experimental panel formed to this safety factor will deliver a successful outcome. Clearly, one successful outcome is not adequate for developing a reliable level of confidence in such a low safety factor design. Statistically, there is no reason why the design might not perform successfully on nine consecutive occasions, before failing on the tenth experimental test.

Two case studies serve to illustrate and reinforce these principles. One of these relates to the tragic collapse at Coalbrook Colliery in 1960, noted in Sect. 4.1 in respect of pillar stability and in Sect. 5.2.2 in respect of the function of interpanel pillars. The section of the mine workings in which the collapse occurred is shown in Fig. 11.16 The collapse was ultimately

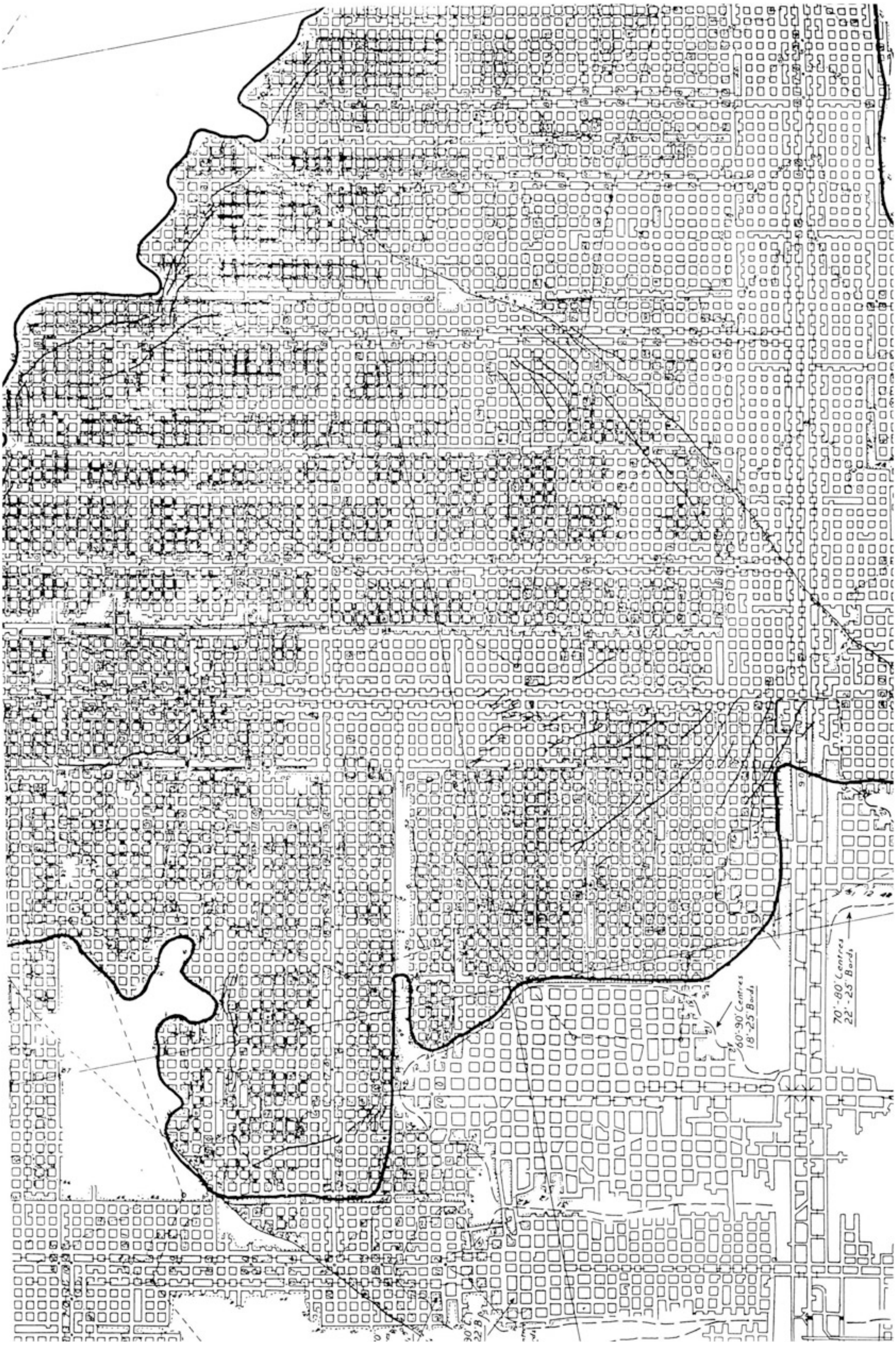
60'-90' Centres
18'-25' Bords
70'-80' Centres
22'-25' Bords
30' C
22' B

**Fig. 11.16** An extract from a mine plan showing, within the black boundary line, the area involved in Coalbrook Colliery pillar collapse

arrested by interpanel pillars and larger size panel pillars. The mine was under pressure to meet increased production targets to satisfy a new contract and had established a number of experimental panels to trial higher percentage extraction mining systems. These trials involved the formation of higher and narrower pillars. The leaving of interpanel pillars was also discontinued and extraction occurred in existing interpanel pillars. A precursory collapse some 3 weeks prior to the event was arrested by interpanel pillars but these only provided temporary protection as they were also in the process of failing.

Bryan et al. (1964) discuss some of the geotechnical aspects of the incident, with the findings of the Inquiry into the disaster presented by Moerdyk (1965). As discussed in Sect. 5.2.2, one of the fundamental findings of the Inquiry was that mining should be carried out in panels surrounded by barriers of unworked coal of dimensions which will limit instability to a single panel in the event of pillar collapse. The importance of regional barrier pillars and interpanel pillars in limiting the extent and consequences of a pillar system failure cannot be overemphasised.

The second case study relates to Fig. 11.17, which shows an experimental panel utilised for evaluating the impact on pillar strength of driving stubs in square pillars. The panel width-to depth-ratio, W/H, was 1.1 and the pillars had a width-to-height ratio, w/h, of 4.4 prior to stubbing. The outcomes of the trial were incorporated into the mine plan shown in Fig. 11.18. This plan also utilized diamond shaped pillars on one side of the production panel in order to minimise the impact of cleat on rib stability. It was also decided not to leave interpanel pillars against adjacent extracted panels.

The mine had previously designed panels to a safety factor of 1.6 using the UNSW power pillar strength formula. Based on the outcomes of the experimental panel and other limited mining operations, it was decided that the design safety factor of the panel shown in Fig. 11.18 could be reduced to 1.2. Fortunately, a change in senior management resulted in mining being halted and an audit being undertaken when the panel had only retreated a distance about equal to depth (W/H = ~1).

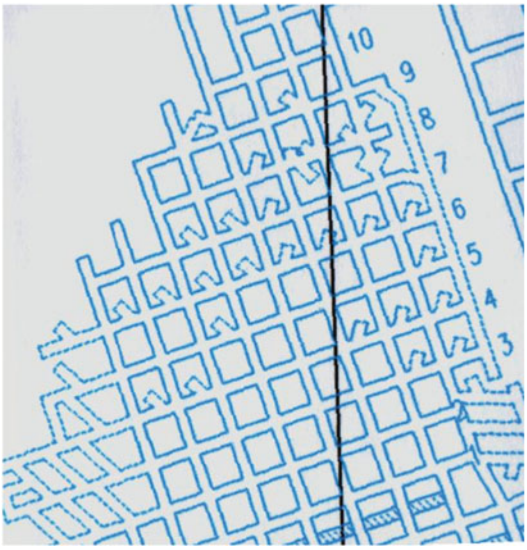

**Fig. 11.17** The layout of an experimental panel used to assess the impacts on pillar system stability of driving stubs in pillars (After Galvin 2002)

The audit revealed that the design as executed had a safety factor of only 1.02, due to deviations in the planned size of the pillar stubs. Closer examination identified an error in spreadsheet computations of pillar stability. When this and the severe deterioration of the acute corners of the diamond shaped pillars that had already occurred were taken into account, the actual safety factor was only 0.8, corresponding to a probability of collapse of 90 %. The mine plan was immediately altered to bring the safety factor back to an acceptable value, with the mine ceasing production as planned soon after the completion of the panel. Some months after mine closure, the experimental panel collapsed in a violent manner, damaging the mine seals.

The case study reinforces a number of principles related to coal pillar system stability, which may be grouped under the headings of experimental panels, operational panels and generally. These are:

Experimental panels

- Experimental panels should be designed to generate similar loading conditions to those that the pillar system will be subjected to if the experimental design is applied to routine production operations.

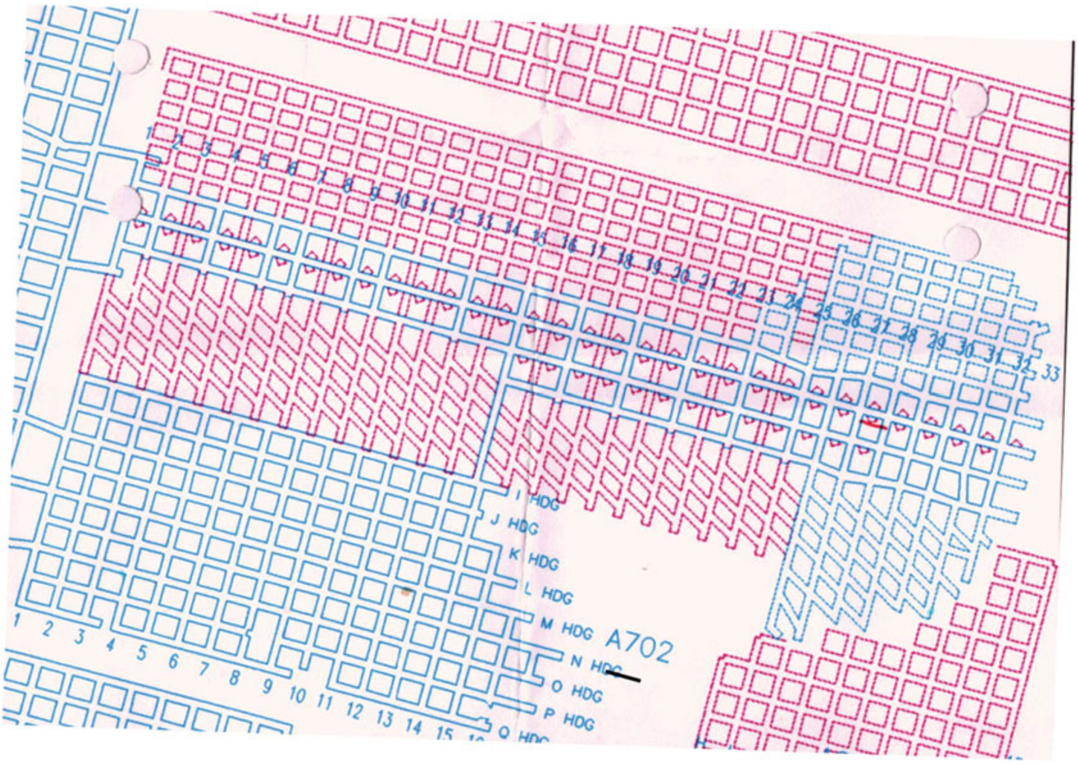

**Fig. 11.18** The production plan incorporating aspects of the experimental panel - actual workings in blue, planned workings in red (After Galvin 2002)

The irregular outline and small width-to-depth ratio of the experimental panel in the case study were not conducive to the panel pillars being subjected to full deadweight loading conditions that were to apply later in the production panels.

- Experimental panels need to be developed in a manner which ensures that the impacts of a change in design can be properly identified and evaluated. The non-systematic and unordered manner in which stubs were developed in pillars in the experimental panel in the case study, compounded by the final shape of the pocketed pillars, created irregular pillar loadings which, at best, could only be sensibly analysed using three-dimensional numerical modelling.
- The assessment of the outcomes from an experimental panel needs to have regard to the fact that the strength of rock can decrease over time. The experimental panel in the case study only failed some 2 years after it was developed.
- One successful outcome from an experimental panel should not be interpreted to mean that all future outcomes will be successful.
- Pillars of small width-to-height ratio can fail violently, generating large windblasts.

Operational Panel

- If the design of an operational panel is premised on the outcomes from an experimental panel, the design needs to adequately reflect the layout of the experimental panel. The pillar layout of the operational panel in the case study did not reflect that of the experimental panel.
- Diamond shaped pillars are particularly prone to rib spall on their acute corners. When pillars are small, this can result in a significant increase in pillar stress.

- The effective width-to-height ratio and, hence, strength of the irregular pocketed coal pillars is not covered by standard pillar strength approaches.
- Irregular shaped portions of a coal pillar should not be considered to have the same average load carrying capacity per unit area (that is, strength) as the remainder of the associated pillars. Rather, the strength of these segments is considerably less. The safety factor of the pocketed pillars in the case study was inflated by assigning the average pillar strength to the remnant sections.
- The pocketing of pillars requires strong operating discipline and control. A small increase in extraction can result in a substantial reduction in the stability of the pillar system, especially when the pillars are already relatively small.

Generally

- Visual observations of pillar system stability can be misleading and should always be supported with knowledge.
- Interpanel pillars are essential controls for catering for ignorance, unknowns, error, and the unplanned.
- Good mine design principles must not be permitted to succumb to economic forces.
- Mine design based on judicious experience should not be interpreted as conservatism.
- Experience demonstrates that statistically determined probabilities apply when taken over a sufficiently long time. Therefore, the limits should not be pushed.

## 11.15 Alternative Rock Bolt Applications

Although the primary function of rock bolts is to pin and reinforce the rock mass, rock bolts are also utilised for a range of other purposes involved with suspending, securing and anchoring equipment such as lifting devices, monorails, conveyor boot ends and conveyor transfer jibs. These applications should be the subject of a change management risk assessment to verify that the rock bolts are fit-for-purpose.

A number of serious incidents and fatalities have been associated with these alternative uses, often because the bolts have been subjected to shock loading and to excessive shear load. The risk is presented by the broken end of a bolt becoming a projectile, by the load suspended by a bolt inducing a fall of ground, and by the unplanned movement of the equipment that the bolt was securing. Risk is associated with all types of rock bolts. However, high tensile bolts usually present a greater risk because they are stiffer and can store very large amounts of energy prior to failure (see Eq. 2.6) and because they then tend to fail in a brittle manner. Design needs to take account of both the axial and the shear strength of the bolts and the axial and transverse loads to which they may be subjected. Fatigue loading may be an issue in applications where the bolts are subjected to repeated and severe cycles of shear loading, such as in the case of bolts used to anchor a boot end to the floor or a coal transfer chute to the roof.

## 11.16 Convergence Zones and Paleochannels

The terms ‘convergence zone’ and ‘seam split’ denote the general area in which two seams coalesce, or alternatively, split. There is a history of very poor ground conditions being associated with convergence zones (reference, for example, Galvin 1996; Frith and Creech 1997; Moodie 2006). Poor ground conditions are also often associated with the flanks of paleochannels in close proximity to the seam horizon. Hanson et al. 2005 report that in Australian underground coal mines these types of zones of disturbance are often associated with:

- channelisation of strata and associated variation in rock mass characteristics and stress distributions where rider seams diverge;

- differential compaction features (often wrongly interpreted as low angle shear zones);
- localised seam thinning;
- increased density of jointing in the immediate roof.

All of these factors can combine to produce highly variable and poor roof conditions in development roadways and secondary extraction panels. Therefore, paleochannels and convergence zones, including zones where a rider seam may converge from the top of a main seam, should be identified on Hazard Plans (see Sect. 12.6.1) and be given careful consideration when laying out mine workings, developing ground support systems and scheduling production rates.

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# Managing Risk in Ground Engineering

# 12

**Abstract**

All stages in the life cycle of a construction in geological materials are characterised by uncertainty. This is because it is neither practical nor economically feasible to fully identify the composition and properties of ground engineering materials. The geological, geomechanical and geotechnical engineering knowledge bases that underpin stability assessment and design are still evolving. Moreover, there is a range of design approaches, each of which has its strengths and its weaknesses, and ground conditions can change over time. A Ground Control Management Plan (GCMP) that is consistent with ISO 31000, the international standard for risk management, provides a basis for safely and effectively managing geotechnical uncertainty.

The philosophy behind a GCMP and the generic structure for a GCMP are presented. A distinction is drawn between the concepts of reducing risk to '*as low as reasonably practicable*' and reducing risk to '*so far as is reasonably practical*'. Matters considered include Risk Assessment Techniques and Processes; Hazard Plans; Trigger Action Response Plans (TARPs); Professional Competencies; Change Management; Auditing of Risk Assessments; Residual Risk; and Monitoring Devices and Strategies. Extracts from a range of GCMPs are presented in the chapter and associated appendices to illustrated aspects of these elements.

Monitoring is integral to the effectiveness of a GCMP, both for avoiding an unwanted event and in managing the consequences of such an event occurring. The chapter concludes with a discussion of instrumentation options and monitoring strategies and a reminder that the most important consideration in ground engineering must always be the safeguarding of health and safety.

**Keywords**

Borescope • Bow tie • Change management • Competency • Consequence • Convergence pole • Event • Extensometer • Failure modes and effects analysis • Fault tree • Flatjack • FMEA • GCMP • Ground control management plan • Hazard • Hazard map • Hazard plan •

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Hydrofracturing • Inclinometer • ISO31000 • Likelihood • Load cell • Microseismic monitoring • Monitoring • Observational method • Piezometer • Probability • Qualitative risk assessment • Quantitative risk assessment • Risk • Risk assessment • Risk management • Seismicity • Shear strip • Sonic probe • Strata failure management plan • Stress measurement • TARP • Telltale • Threat • Tiltmeter • Trigger action response plan • Wire extensometer • WRAC

## 12.1 Introduction

Risk management refers to the architecture (principles, framework and processes) for managing risks effectively. The process comprises the systematic application of management policies, procedures and practices to the activities of communicating, consulting, establishing the context, and identifying, analysing, evaluating, treating, monitoring and reviewing risk (ISO 31000 2009). The core framework for risk management has been presented in Chap. 1, with the term *risk* noted to be a combined measure of the likelihood of an event occurring and the consequences should it occur. In the case of an event with adverse outcomes, the source of potential harm is referred to as a *hazard* and the triggers which could cause the hazard to materialise are referred to as *threats*.

The life cycle of a construction in the ground is no different to that of other engineering structures, consisting of pre-feasibility, feasibility, design, as-built, maintenance, and decommissioning. All stages of this cycle are characterised by uncertainty. In particular:

- it is not practical or economically feasible to fully identify the composition and properties of ground engineering materials, which can be variable, complex and prone to incorrect interpretation;
- the geological, geomechanical and geotechnical engineering knowledge bases that underpin stability assessment and design are still evolving;
- there is a range of design approaches, each of which has its strengths and its weaknesses; and
- ground conditions can change over time.

Hence, the ground engineer is faced with uncertainty and choices. The eminent soil mechanics engineer, Karl Terzaghi, advised that '*the geotechnical engineer should apply theory and experimentation, but temper them by putting them into the context of the uncertainty of nature. Judgement enters through engineering geology*' (Palmstron 1996). Malan and Napier (2011) cautioned that '*in rock engineering, over time, initial assumptions and interim solutions become entrenched as common practice and the original assumptions are rarely revisited or questioned*'.

There is still considerable debate around the relative merits of empirical, analytical and numerical methods. In addressing this issue, Suorineni (2014) noted that '*users of empirical methods need to know the underlying assumptions, database limits and inherent hidden risks that are often unmentionable for their satisfactory use.*' Suorineni also noted variously that '*non-believers in empirical methods argue that these approaches never capture the physics of the problem and therefore we never understand such problems*'. On the other hand, '*the empiricists believe the complexity of the rock mass and its interaction with structures developed in it are implicitly accounted for in these methods to accommodate our ignorance and all the inherent uncertainties. It is generally accepted that most rock mechanics problems fall in the data limited region (Starfield and*

*Cundall 1988). Data limited problems are difficult to adequately validate in analytical methods.*'

Against this background, the reality is that ground control remains a mix of art and science, relying heavily on judgements which should be premised on knowledge, skill and experience. Risk management provides a platform for dealing with this pervasive uncertainty.

The international standard for risk management (ISO 31000 2009) refers to uncertainty in terms of **likelihood**. As discussed in Sect. 2.7.5, likelihood is a statistical concept that is concerned with inferring properties about a population based on a random sample from the population. On the other hand, **probability** is concerned with the chance of achieving an outcome from sampling a population about which everything is known. In geotechnical engineering, the distinction between the two terms is often lost. This applies to risk management. While the term, 'likelihood', is the more appropriate term when evaluating risk associated with geotechnical uncertainty, it is interchanged with the term 'probability' in this text, consistent with what has become mining industry custom and practice.

## 12.2 Ground Control Management Plan

### 12.2.1 Basis for a Ground Control Management Plan

Risk is associated with a wide range of business activities and has implications for occupational health and safety (OH&S), the environment, community, government relations, litigation, business performance, corporate reputation and industry reputation. Hence, it is not uncommon for organisations to have 'enterprise wide' risk management policies, standards and procedures, with OH&S assuming the highest priority. As ground control is a core risk in underground coal mining, it now generally features strongly for reasons of safety and business performance in enterprise risk management within major coal mining companies.

Ground control also features strongly, both directly and indirectly, in legislative requirements and approval conditions relating to health, safety and environmental protection in most developed coal producing countries. At the highest level, it is encapsulated in an overarching requirement of OH&S style legislation for an employer not to expose an employee to an unacceptable level of risk in the workplace. The standard to be achieved in this regard varies with community expectations which, in turn, vary from culture to culture, country to country and over time. Risk assessment in ground engineering has evolved from the concept that risk should be reduced to a level that is "As Low as Reasonably Practicable" (ALARP). Since 2011, some legal jurisdictions have required risk to be reduced to "So Far as is Reasonably Practicable" (SFARP), which is a more onerous standard as discussed by Robinson (2014).

Irrespective of which standard is applied, both turn on the meaning of 'reasonably practicable'. According to Justice Marks, there are five elements to the test for what constitutes 'reasonably practicable' (Marks et al. 2013). These are all particularly relevant to the practice of ground engineering and are:

1. The likelihood of the hazard or risk occurring.
2. The degree of harm that might result.
3. What the duty holder ought reasonably to know about the risk and the ways to eliminate it.
4. The availability and suitability of ways to eliminate or minimise the risk.
5. The cost associated with available ways of eliminating or minimising the risk, with the comparison being with the risk and not with the impact on the business.

The owner of an enterprise, as well as manufacturers, suppliers and contractors who have involvement in the conduct of that enterprise, are required to control risks by providing employees with:

- a safe place of work;
- fit-for-purpose equipment;
- a safe system of work;

- adequate information, instruction and training; and
- effective supervision.

Some countries have complementary or standalone legislation specific to the mining sector which stipulates that because ground control presents a risk of multiple fatalities, it must be controlled through the development of a specific management plan premised on risk assessment. Appendix 13 provides examples of these requirements in Australia. The requirement for risk assessment is not confined to legislation. Manly organisations have risk assessment and management standards embedded in their corporate policies and procedures and apply them across the whole business enterprise. This includes both small domestic mining companies and very large global mining organisations.

Leading practice in both managing enterprise risk and in regulating workplace safety requires a mine owner to develop a Ground Control Management Plan as a component of a Mine Safety Management System (MSMS) that is premised on the principles of risk management espoused in ISO 31000 (2009). Enterprise and regulatory risk management performance requirements are generally integrated along the lines shown in Fig. 12.1.

A Ground Control Management Plan (GCMP), which may be known by a range of names such as Strata Control Principal Hazard Management Plan (SCPHMP) and a Strata Failure Management Plan (SFMP), is required to have regard to all legislative requirements, standards, codes of practice, industry guidelines, safety alerts, published scientific and technical literature and other guidance material, and to be supported with safe operating procedures (SOP) and standard working procedures (SWP). A list of some relevant resources is provided in Appendix 14.

### 12.2.2 Structure of a Ground Control Management Plan

Whilst there is considerable variation in the format of Ground Control Management Plans, they are usually structured around the following list of elements. These mirror the guideline for preparing a Mine Safety Management System and Principal Hazard Management Plans (Qld Dept. Mines and Energy 1996), a précis of which is presented in Appendix 15.

1. Scope
2. Geological and Geotechnical Data Base and Models
3. Design
4. Identified Hazards
5. Control Procedures
6. Roles and Responsibilities
7. Required Resources
8. Trigger Action Response Plan
9. Communications
10. Training
11. Corrective Action
12. Review
13. Audit
14. Document Control
15. Records

An example of a ground control management process that encapsulates most of these elements is shown in Fig. 12.2. In this particular case, ‘document control’ and ‘records’ are captured in the enterprise wide risk management procedures. In preparing a Ground Control Management Plan, it is advisable to map the elements of the plan against both enterprise standards and regulatory requirements to verify compliance. The process should be consistent with the universal core functions of management to Plan, Do, Check, and Act.

### 12.2.3 Competencies

Competencies define the knowledge, skills and attitudes acquired by individuals through appropriate formal education and experience (Turner 2011). It was noted in Chap. 1 that the practice of ground engineering relies on a range of interdisciplinary and interdependent strands, with terminology relating to these evolving in a fairly loose manner and resulting in confusion and debate

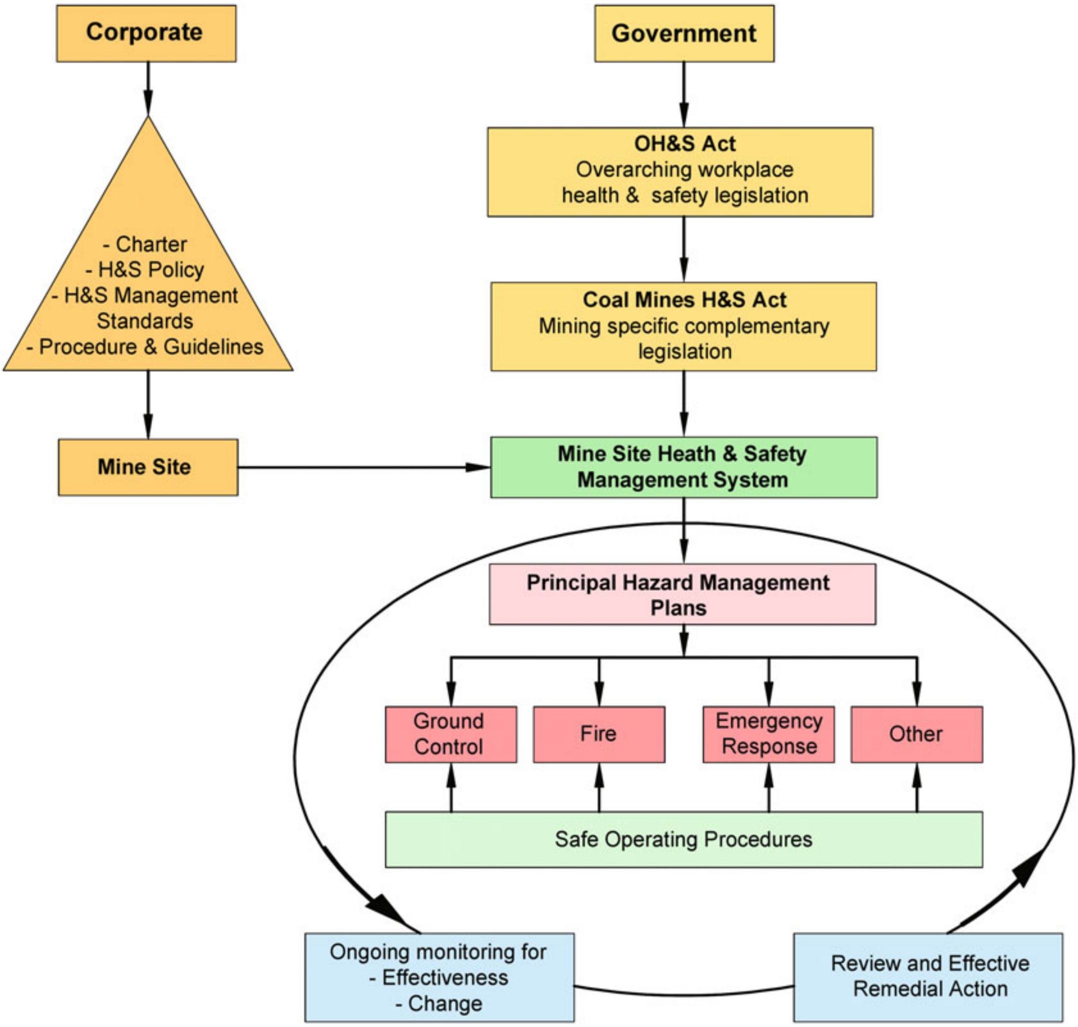

Corporate
- Charter
- H&S Policy
- H&S Management Standards
- Procedure & Guidelines
Mine Site
Government
OH&S Act
Overarching workplace health & safety legislation
Coal Mines H&S Act
Mining specific complementary legislation
Mine Site Heath & Safety Management System
Principal Hazard Management Plans
Ground Control
Fire
Emergency Response
Other
Safe Operating Procedures
Ongoing monitoring for
- Effectiveness
- Change
Review and Effective Remedial Action

**Fig. 12.1** Framework showing how enterprise and regulatory risk management requirements for ground control are generally integrated in a risk management culture

concerning professional competencies and roles. Eminent persons in the field have come from a range of disciplines including civil engineering, mining engineering, mechanical engineering, structural engineering, geotechnical engineering, engineering geology and geophysics.

Mine operators face a dilemma when endeavouring to ensure and verify, consistent with risk management principles and statutory requirements, that persons engaged in the practice of ground engineering are competent. Unfortunately, there are experiences of mine operators only becoming aware as a result of investigations and prosecutions over serious and fatal accidents, that the tertiary qualifications of employees and consultants, their registration of chartered professionals and their membership of learned societies do not always qualify them as competent in the geotechnical roles they are engaged in. The issue of competency is therefore discussed in some detail to an effort to reduce this risk.

There have been a number of notable attempts to define the competencies of those involved in ground engineering and to foster collaboration, rather than competition, between practitioners of differing academic and practical backgrounds. In 1999, a British working group proposed a single

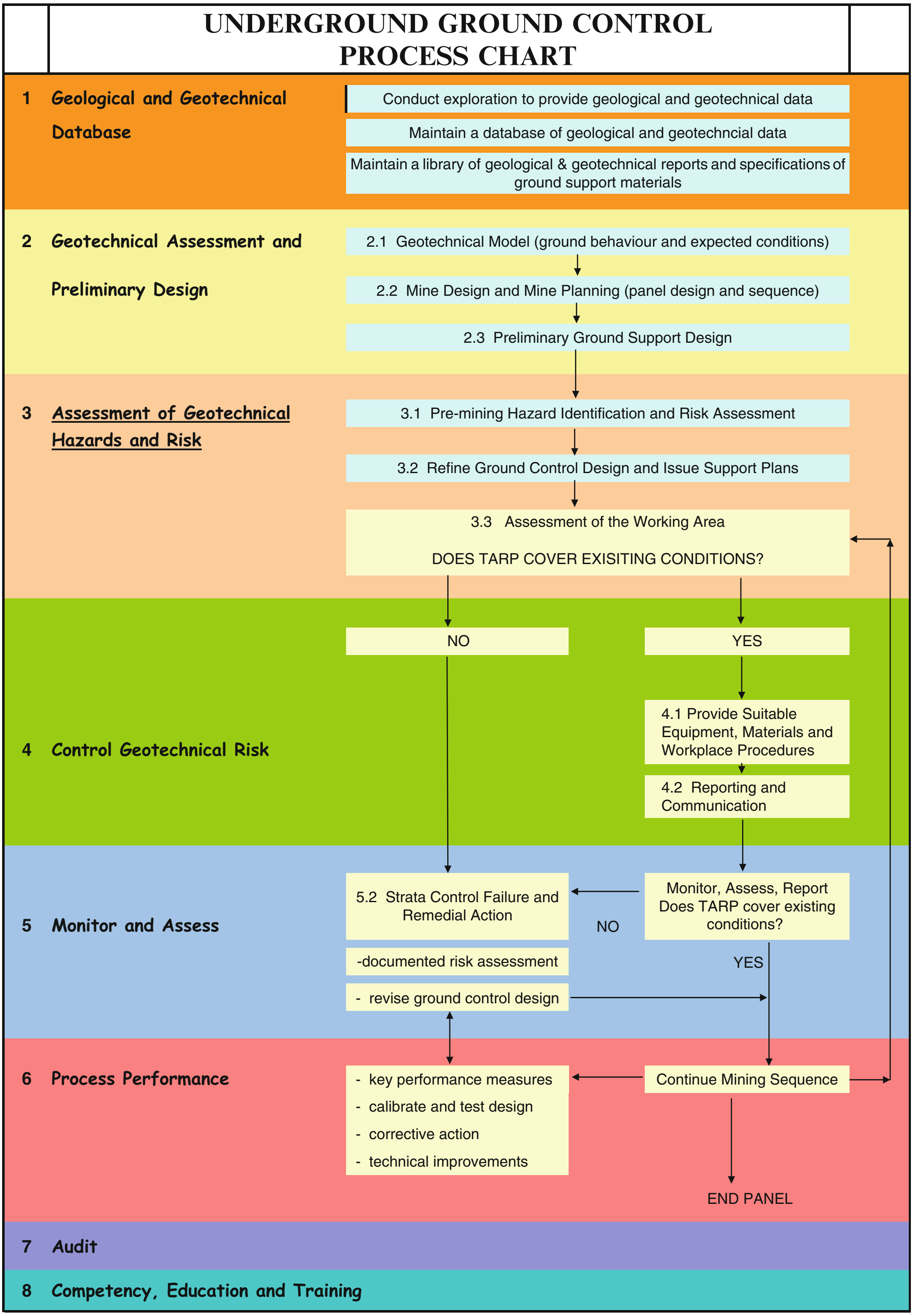

UNDERGROUND GROUND CONTROL PROCESS CHART
1 Geological and Geotechnical Database
Conduct exploration to provide geological and geotechnical data
Maintain a database of geological and geotechncial data
Maintain a library of geological & geotechnical reports and specifications of ground support materials
2 Geotechnical Assessment and Preliminary Design
2.1 Geotechnical Model (ground behaviour and expected conditions)
2.2 Mine Design and Mine Planning (panel design and sequence)
2.3 Preliminary Ground Support Design
3 Assessment of Geotechnical Hazards and Risk
3.1 Pre-mining Hazard Identification and Risk Assessment
3.2 Refine Ground Control Design and Issue Support Plans
3.3 Assessment of the Working Area
DOES TARP COVER EXISITING CONDITIONS?
NO
YES
4 Control Geotechnical Risk
4.1 Provide Suitable Equipment, Materials and Workplace Procedures
4.2 Reporting and Communication
5 Monitor and Assess
5.2 Strata Control Failure and Remedial Action
NO
Monitor, Assess, Report Does TARP cover existing conditions?
-documented risk assessment
YES
- revise ground control design
6 Process Performance
- key performance measures
- calibrate and test design
- corrective action
- technical improvements
Continue Mining Sequence
END PANEL
7 Audit
8 Competency, Education and Training

**Fig. 12.2** An example of a ground control management process chart

learned society for geotechnical engineering in the UK, to be known as the British Geotechnical Association (Anonymous 1999).

In 2002, a Joint European Working Group of the three learned international societies in the broader field of ground engineering was established in response to ongoing debate concerning the particular contributions and responsibilities of engineering geologists and geotechnical engineers in solving ground control problems, and the differing professional definitions and accreditation rules that exist for these professions. These learned societies were the International Society for Rock Mechanics (ISRM), the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) and the International Association for Engineering Geology (IAEG). The working group issued reports in 2004 and 2008 that have resulted in a clearer definition of roles and responsibilities (Bock et al. 2004; JEWG 2008). However, they do not provide a definitive basis for determining the competency of those involved in the very broad scope of ground engineering activities associated with mining.

In developing guidelines for managing the risk of landslides, the Australian Geomechanics Society (AGS, which represents ISRM in Australia) defined a practitioner as '*A specialist Geotechnical Engineer or Engineering Geologist who is degreed qualified, is a member of a professional institute and who has received chartered professional status*' (Australian Geomechanics Society 2007). This definition still leaves open the question of what degree program qualifies someone as a 'geotechnical engineer', with geotechnical engineering being a core component of a range of engineering programs, most notably civil engineering and mining engineering, as well as being offered as a standalone degree program at some institutions.

The American Society of Civil Engineers (ASCE) has defined the domain of knowledge and experience considered to be essential for a qualified civil engineer according to 24 competencies. Turner (2011) reported that these were being used as a model by The Joint Technical Committee on Education and Training (JTC-3), established in 2006 under the umbrella of the Federation of International Geo-engineering Societies with the mandate to develop a 'State-of-the-Art Report on Education and Training in Engineering Geology, Rock Mechanics, Soil Mechanics and Geotechnical Engineering'.

The South African mining industry endeavoured to address the issue of competency in ground engineering in the early 1980s by introducing elementary and advanced certificates of competency in rock engineering administered through the Chamber of Mines of South Africa (COMSA). In the late 1990s, the Australian coal industry introduced a range of pre-requisite competencies in strata management for statutorily appointed production supervisors and managers of mining engineering (mine managers). These now form the framework for some postgraduate programs in aspects of ground engineering.

However, none of these approaches guarantee that practitioners in ground engineering, including consultants, have an underpinning degree in engineering or appropriate practical experience to support their advice. Similarly, in many cases membership of a learned society no longer provides assurance that the member holds a graduate qualification in the core discipline that the society represents. Unfortunately, some mine operators have only become aware of this following an incident when they have been called to account for the competencies of those they employed and engaged for consulting advice. The lack of clarity regarding competencies is also unhelpful to mine operators when they are faced with resolving conflicting advice from consultants, especially when this advice is founded on opaque and fiercely defended proprietary approaches.

Against this background, the uncertainty that surrounds competency can be factored into risk management by:

- not placing a blind reliance on 'expert' external advice being an effective control;
- requiring design procedures and expert advice to be subjected to robust independent third party peer review, commensurate with the risk arising out of unreliable advice; and

- applying a quantitative approach to risk assessment to assess the potential impact of advice should it prove to be incomplete, inappropriate or inaccurate.

## 12.3 Risk Analysis Foundations

Ground control risk management extends well beyond the traditional scope of controlling falls of ground. It has implications for managing a range of other risks associated with underground coal mining, such as spontaneous combustion, inrush, slips and trips of mine personnel, and impacts on subsurface and surface features. Hence, ground engineering practitioners and mining engineers need to work closely in planning, designing and optimizing mine layouts. The process commences with collection and collation of data to define geotechnical domains, failure mechanisms and preliminary designs. This should encompass:

- Local assessment of roof and floor lithology, typically to at least 10 m into the roof and 5 m into the floor (and preferably to twice that distance), including:
  - rock types;
  - bed thickness;
  - variability in thickness;
  - bedding angles;
  - strength data.
- Coal seam characteristics and integrity, including:
  - ply structure and composition;
  - parting composition, thickness, swelling and weathering characteristics;
  - material properties/friability;
  - floor dip, strike and contours;
  - proneness to spontaneous combustion;
  - gas composition, pressures, volumes and desorption characteristics;
  - cleat intensity and direction;
  - coal quality.
- Local structural features, including:
  - joint sets;
  - low angle bedding planes (feather edges);
  - bedding place shears;
  - slickensides or ’greasy backs’;
  - pot arses;
  - minor faulting;
  - floor depressions (swillies).
- Regional assessment of lithology, including thickness, depth and composition of:
  - massive stratum;
  - aquicludes;
  - unconsolidated sediments (alluviums, sand beds etc).
- Regional structural features, such as:
  - major faults, dykes, sills, shear zones;
  - folding;
  - surface lineaments.
- Stress regime, including:
  - local and regional regimes;
  - primitive and mining induced stresses;
  - quantitative stress measurements;
  - mapping of evidence of stress magnitude and direction;
  - assessment of impacts of rapid changes in topography.
- Groundwater regime, focusing on:
  - aquifers;
  - aquitards;
  - aquicludes;
  - water pressures and flow rates;
  - material properties (permeability, fracture networks);
  - water quality - pre and post mining.
- Mining conditions, with consideration of:
  - state of any existing workings (in-seam, above, and below), including dimensions, effectiveness of support and reinforcement systems, spalling and falls of ground, floor heave, water ingress, preferential directions for deformation etc;
  - influence of surrounding workings (adjacent, above, and below) on stress, deformation, inrush potential, spontaneous combustion, noxious and flammable atmospheres;
  - identification of dominant behaviour and failure mechanisms.
- Sub-surface and surface constraints, including:
  - natural environment (aquifers, streams, cliffs, swamps, indigenous heritage etc);

  - built environment (sub-surface and surface);
  - amenity (dust, noise, traffic, visual impacts).

These data provide the basis for constructing geological, geotechnical and ground models (Fig. 1.2) and for classifying ground conditions into distinct geotechnical domains. Mine designs are then prepared and subjected to an iterative process of risk assessment and refinement. This may result in the generation of a second set of domains based on operational factors such as mining method, functional requirements of excavations, and life of excavations. Hence, it is possible to have different levels of ground support and monitoring in similar geotechnical conditions and, conversely, different geotechnical conditions with similar levels of ground support. Openings such as those that are travelled regularly by personnel are critical ventilation and emergency corridors, or are intended to service the life of mine may warrant a higher standard of ground support and monitoring.

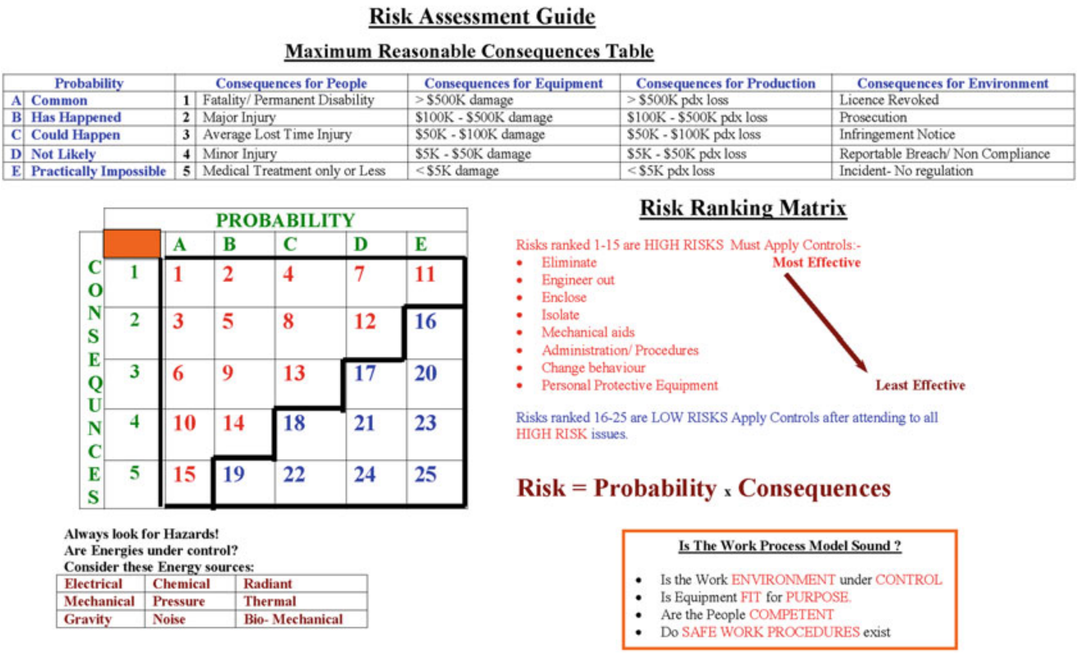
## 12.4 Types of Risk Assessment

Risk assessment can be undertaken in a qualitative manner (where likelihood is not linked to probability), semi-quantitative (where likelihood is linked to an indicative probability), or quantitative manner (where events are assigned a probability of occurrence). The advice of the Australian Geomechanics Society (2007) in respect of managing the risk of landslides is relevant to ground engineering in general, being that a qualitative or semi-quantitative analysis may be used:

- as an initial screening process to identify hazards and risk which require more detailed consideration and analysis;
- when the level of risk does not justify the time and effort required for more detailed analysis; or
- where the possibility of obtaining numerical data is limited such that a quantitative analysis is unlikely to be meaningful or may be misleading.

The following basic risk assessment techniques find extensive application to ground control, with further information available in *MDG 1010 - Risk Management Handbook for the Mining Industry* (MDG-1010 2011) and a range of other publications:

- Workplace Risk Assessment and Control (WRAC). This is a qualitative technique that relies on a suitably qualified team identifying hazards, assessing likelihood and consequences, scoring and ranking risks, identifying risk reduction measures, and determining residual risk. Figure 12.3 shows a summary guide that is commonly used for this and a range of other risk assessment processes. The system revolves around what level of risk an organisation is prepared to accept, with leading practice being to refer residual risks to progressively higher levels within management, commensurate with the residual risk score.
- Failure Modes and Effects Analysis (FMEA). This is also a qualitative technique and is based on identifying failure modes and associated outcomes as a basis for either deciding to prevent failure or else to detect and respond to failure in a manner that limits its impacts.
- Fault Tree Analysis. This is a quantitative technique that is used to analyse, understand and display the logical structure of events and situations which can lead to an undesired outcome. The process commences with identifying the undesired outcome, or '*top event*' and working backwards to identify events and construct a logic tree showing how the events either in their own right (this or that) or in combination (this and that) could lead to the top event. The process continues until the base events are sufficiently simple and understandable to be regarded as 'root causes', as shown in the example presented in Fig. 12.4. By assigning a likelihood to each event, the risk of a top event can be quantified.



**Risk Assessment Guide**

**Maximum Reasonable Consequences Table**

| Probability | | Consequences for People | | Consequences for Equipment | Consequences for Production | Consequences for Environment |
|---|---|---|---|---|---|---|
| A | Common | 1 | Fatality/ Permanent Disability | > $500K damage | > $500K pdx loss | Licence Revoked |
| B | Has Happened | 2 | Major Injury | $100K - $500K damage | $100K - $500K pdx loss | Prosecution |
| C | Could Happen | 3 | Average Lost Time Injury | $50K - $100K damage | $50K - $100K pdx loss | Infringement Notice |
| D | Not Likely | 4 | Minor Injury | $5K - $50K damage | $5K - $50K pdx loss | Reportable Breach/ Non Compliance |
| E | Practically Impossible | 5 | Medical Treatment only or Less | < $5K damage | < $5K pdx loss | Incident- No regulation |

PROBABILITY

CONSEQUENCES

| | A | B | C | D | E |
|---|---|---|---|---|---|
| 1 | 1 | 2 | 4 | 7 | 11 |
| 2 | 3 | 5 | 8 | 12 | 16 |
| 3 | 6 | 9 | 13 | 17 | 20 |
| 4 | 10 | 14 | 18 | 21 | 23 |
| 5 | 15 | 19 | 22 | 24 | 25 |

**Always look for Hazards!**
**Are Energies under control?**
**Consider these Energy sources:**

| | | |
|---|---|---|
| Electrical | Chemical | Radiant |
| Mechanical | Pressure | Thermal |
| Gravity | Noise | Bio- Mechanical |

**Risk Ranking Matrix**

Risks ranked 1-15 are HIGH RISKS Must Apply Controls:-

- Eliminate
- Engineer out
- Enclose
- Isolate
- Mechanical aids
- Administration/ Procedures
- Change behaviour
- Personal Protective Equipment

Risks ranked 16-25 are LOW RISKS Apply Controls after attending to all HIGH RISK issues.

**Risk = Probability x Consequences**

**Is The Work Process Model Sound ?**

- Is the Work ENVIRONMENT under CONTROL
- Is Equipment FIT for PURPOSE.
- Are the People COMPETENT
- Do SAFE WORK PROCEDURES exist

**Fig. 12.3** An example of a summary guide for qualitatively scoring risk on the basis of likelihood and consequence

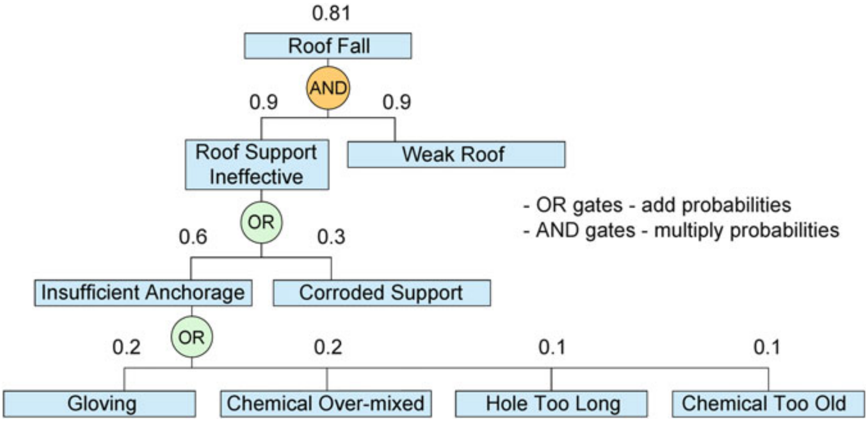
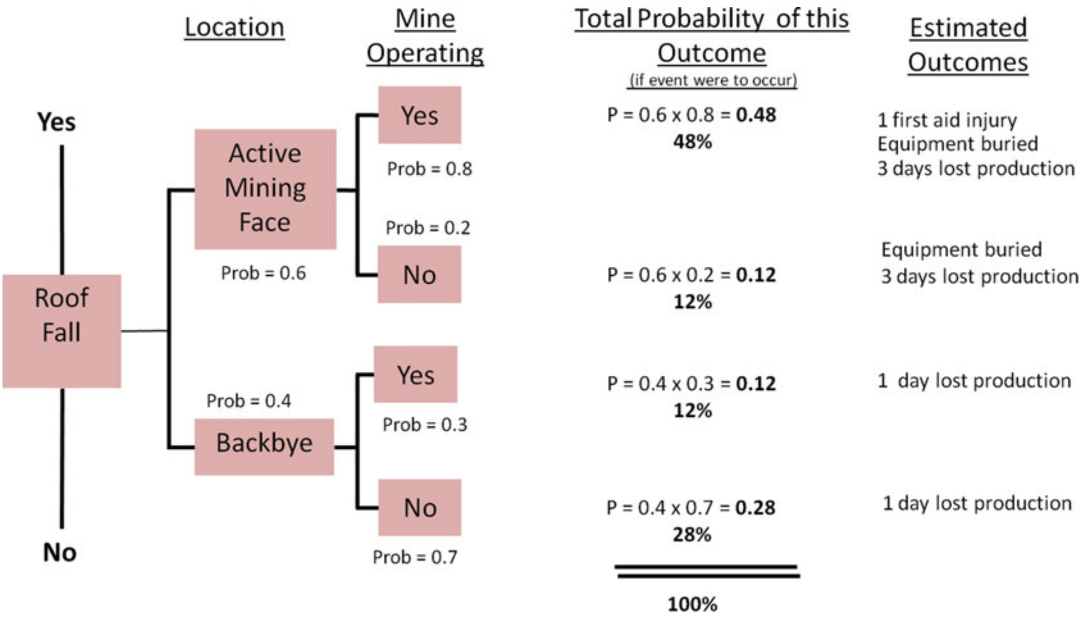


**Fig. 12.4** A simplified example of a quantitative fault tree risk assessment

- A fault tree is particularly useful when the logical structure of the causes of a top event are not immediately clear and when these causes, in turn, have various root causes, preventative measures and response options. It provides a structured approach to developing a good understanding of how causes and safeguards are logically linked; to recognising weaknesses in the safeguards; and to identifying the most appropriate risk reduction measures. However, it can be difficult to identify some probabilities, with reliance having to be placed on subjective judgements. In these situations, sensitivity analysis may be applied to judgements to evaluate the error range they introduce into the probability prediction of the top event.
- Event Trees Analysis. This is a quantitative technique for estimating the range of possible outcomes and their likelihood arising from an event. The process commences with postulating the unwanted event and working


Location
Mine Operating
Total Probability of this Outcome
(if event were to occur)
Estimated Outcomes
Yes
No
Roof Fall
Active Mining Face
Prob = 0.6
Backbye
Prob = 0.4
Yes
Prob = 0.8
Prob = 0.2
No
Yes
Prob = 0.3
No
Prob = 0.7
P = 0.6 x 0.8 = 0.48
48%
P = 0.6 x 0.2 = 0.12
12%
P = 0.4 x 0.3 = 0.12
12%
P = 0.4 x 0.7 = 0.28
28%
100%
1 first aid injury
Equipment buried
3 days lost production
Equipment buried
3 days lost production
1 day lost production
1 day lost production

If the likelihood of a roof fall in a mine is 50% per year, then the annual likelihood of a roof fall causing:

- A first aid injury = 0.5 x 0.48 = 0.24 or 24%
- Equipment damage = 0.5 x (0.48 + 0.12) = 0.30 or 30%

**Fig. 12.5** A simplified example of a quantitative event tree risk assessment

forward to identify the factors that influence possible outcomes. Probabilities are assigned to alternative outcomes as branches are developed, as in the example shown in Fig. 12.5. Branch tips identify possible consequences and their likelihoods. This enables identification of the worst credible outcome, the least severe outcome, a weighted best estimate, and situations which have the most effect on the outcome. The main limitations with the technique relate to difficulties in defining some alternative outcomes and to estimating the probability of each alternative outcome.

Fault trees and event trees provide a structured and auditable approach for the use of expert judgement and likelihood assessment. They permit sensitivity analysis of input data and give insight into where to focus design effort, control, monitoring and review activities. A fault tree and an event tree can be combined into a bow tie diagram to provide a powerful structured graphical representation of a risk assessment, with the fault tree identifying the causes of a top event for which controls have to be developed, and the event tree identifying the consequences of the top event for which emergency responses and contingencies have to be developed. This process is shown in Fig. 12.6 using the examples of a fault tree and an event tree presented earlier. Bow tie diagrams are particularly useful for breaking down complex risk into its contributory elements and for summarising and communicating the overall risk analysis process.

## 12.5 Risk Assessment Process

### 12.5.1 Context

Risk assessment can be undertaken for a range of purposes, by a variety of stakeholders, at different stages in a life cycle, and at different

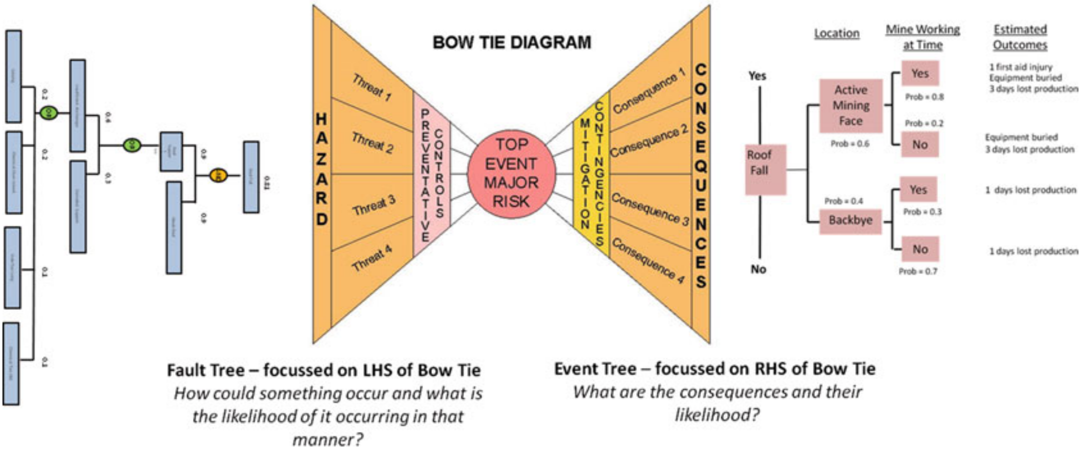

BOW TIE DIAGRAM
HAZARD
Threat 1
Threat 2
Threat 3
Threat 4
PREVENTATIVE CONTROLS
TOP EVENT MAJOR RISK
MITIGATION CONTINGENCIES
Consequence 1
Consequence 2
Consequence 3
Consequence 4
CONSEQUENCES
Location
Mine Working at Time
Estimated Outcomes
Yes
No
Roof Fall
Active Mining Face
Prob = 0.6
Backbye
Prob = 0.4
Yes
Prob = 0.8
Prob = 0.2
No
Yes
Prob = 0.3
No
Prob = 0.7
1 first aid injury
Equipment buried
3 days lost production
Equipment buried
3 days lost production
1 days lost production
1 days lost production
Fault Tree – focussed on LHS of Bow Tie
How could something occur and what is the likelihood of it occurring in that manner?
Event Tree – focussed on RHS of Bow Tie
What are the consequences and their likelihood?

**Fig. 12.6** A bow tie diagram integrated with a fault tree and an event tree to graphically represent the structure of the risk assessment process

strategic and tactical levels. It is important, therefore, that any risk assessment commences with setting the context for the benefit of the participants and end-user and for future reference.

## 12.5.2 Team Composition

The quality of a risk assessment is highly dependent on the composition of the risk assessment team, which needs to be selected on the basis of the nature of the risk being assessed. It is always important that the team is led by a competent facilitator and comprises an appropriate mix of individuals who bring in-house knowledge and skill, external technical expertise, practical experience and corporate memory to the risk assessment. Poor team composition and facilitation can result in:

- a lack of relevant knowledge, skill and experience;
- failure to identify all significant hazards;
- dominant member rules;
- ‘group think’, where the majority rule;
- poor knowledge of incident frequencies;
- omission of credible incidents;
- unjustified optimism.

Risk assessment teams comprised of only local decision makers are prone to:

- Normalise the risk. That is, over a period of time, a certain level of risk comes to be accepted as the norm and so is disregarded. The absolute magnitude of risk is discounted, with the result that risk assessment tends to be focused on incremental risk and not the total risk.
- Display the ‘Not Invented Here’ Syndrome: That is, there is a closed minded attitude to outside ideas and experiences.
- Foster an environment where ‘you don’t know’ and, of more concern, ‘you don’t know you don’t know’.

It is important to consult with the workforce when developing a Ground Control Management Plan and to have them represented on associated risk assessments. Their practical insight into ground behaviour can be invaluable, albeit that they may have no appreciation of the mechanisms involved. Involvement of senior line management is also important. Firstly, its demonstrates commitment and visible felt leadership to achieving a healthy and safe workplace. Secondly, it gives management an understanding of the risks that they own and that they are responsible and accountable for accepting and managing.

### 12.5.3 Controls

Risk analysis and ranking provide the bases for identifying risks which are most in need of reduction and risks which need careful ongoing management. The hierarchy for treating risks is:

1. Eliminate the hazard
2. Substitute the hazard with one having a lower, acceptable level of risk
3. Isolate the hazard from people and things at risk
4. Develop engineered controls, to prevent or restrict energy release and to limit exposure to energy releases
5. Develop administrative controls, to avoid initiating or coming into contact with energy release
6. Utilise personal protective equipment, to restrict the consequences of coming into contact with energy release

Controls which eliminate a hazard, reduce the potential magnitude of uncontrolled energy release, or place physical barriers around a potentially damaging energy source are referred to as ‘hard’ controls because they do not rely on human behaviour to actively restrict the release of damaging energies. Controls of a procedural or administrative nature, such as operating procedures, training, and personal protective equipment, are referred to as ‘soft’ controls because they do not reduce the potential level of energy release. Rather, their effectiveness in preventing the release of energy and/or limiting its potential negative consequences is highly dependent on human behaviour, which is notoriously unreliable.

Having identified controls, it is important that they are risk assessed in their own right to verify their feasibility and their likely effectiveness so as to understand and appreciate residual levels of risk. This process is also valuable for developing standard operating procedures and safe working procedures to support the Ground Control Management Plan. An example of a safe working procedure is presented in Appendix 7.

### 12.5.4 Other Process Considerations

The various standards and guidelines pertaining to risk assessment provide a comprehensive range of other factors that should be considered in the risk assessment process. The more important and fundamental include:

- All assumptions and reasoning that underpin a risk assessment should be documented so that there is an audit trail and their validity can be confirmed in practice.
- All persons participating in the risk assessment process should sign off on the final document.
- Risk assessment is of limited value, and may even result in a more hazardous situation, if the controls identified for managing the risk are not feasible or adequately resourced.
- Risk treatments should be supported with an action plan and an implementation plan and these should be in place prior to start up.
- Risk assessment reports should rank hazards both in terms of their overall risk score and their consequence rating. Consequence rating addresses the dilution of risk ratings through over-optimistic estimations of unlikelihood and alerts management to risks which may be considered totally unacceptable irrespective of their estimated unlikelihood. This could include, for example, multiple fatality events, serious chemical spills and environmental damage.
- The effectiveness of the risk management premised on risk assessment depends on the subsequent provision of fit-for-purpose equipment; supporting management plans; standard operating procedures and safe working procedures; training and competency assessment in these plans and procedures; and effective supervision.

- An internal process is required to ensure that the identified controls are implemented and followed.

## 12.6 Implementation

It is essential when implementing any risk management plan that it is supported by monitoring to verify the robustness of the risk assessment process; to confirm compliance with design and associated working procedures; and to detect and respond to conditions and circumstances that are different to those on which the plan was based or which subsequently change. Reporting of significant hazards, near-misses (near-hits) and incidents by all stakeholders are particularly valuable monitoring actions, especially in the case of a Ground Control Management Plan. The technical aspects of monitoring ground behaviour are discussed in more detail in Chap. 12. Hazard Plans and Trigger Action Response Plans (TARPS) are also very important elements associated with implementing a Ground Control Management Plan.

### 12.6.1 Hazard Plans

Hazard plans are a proactive risk management tool that show the range, location and extent of known and predicted ground control hazards and threats. They are also used for other purposes such as production forecasting, scheduling, equipment selection, and designing mine layouts (Galvin et al. 1995). Hazard plans are based on exploration data, geophysical surveys and underground mapping and observations and are live documents that need to be regularly updated to maximize their effectiveness in pre-empting and creating awareness of hazards. An extract from a Hazard Plan is shown in Fig. 12.7.

Features that should be assessed in the preparation of a Hazard Plan include:

- geological structures – type, direction, characteristics;
- seam rolls/swillies;
- channels of more massive and competent superincumbent strata;
- roof sag and guttering;
- adjacent sites of past incidents of spontaneous combustion;
- inferred stress direction;
- remnant pillars and goaf edges in surrounding seams;
- abutment stress zones;
- potential and actual sites of stored fluids (water, gas) and materials that flow when wet that could be impacted by strata movement;
- tensile fractures;
- excessive water cracks and water drippers;
- stressed ground support systems;
- significant rib failure;
- over-width drivage;
- off-centre drivage;
- changes in seam characteristics, for example, thinning/thickening, bands, hardness, cleat density and direction;
- standard of installed support – deficient, not to design, corroded;
- joint swarms;
- roof fretting and scaling;
- sites of existing falls of ground;
- loss of horizon control; and
- floor heave.

### 12.6.2 Trigger Action Response Plans

The unknowns and limitations associated with processes for developing geological, ground and geotechnical models, such as the process depicted in Fig. 1.2, mean that ground control risk management is unavoidably reactive to some extent. This is managed through Trigger Action Response Plans (or TARPs). A Trigger Action Response Plan is a plan designed to prevent a hazard from developing into a ‘top event’ by proactively identifying precursory signs; assigning a series of staged threshold limits, or trigger levels, to each precursor; and specifying responses and response accountabilities when a

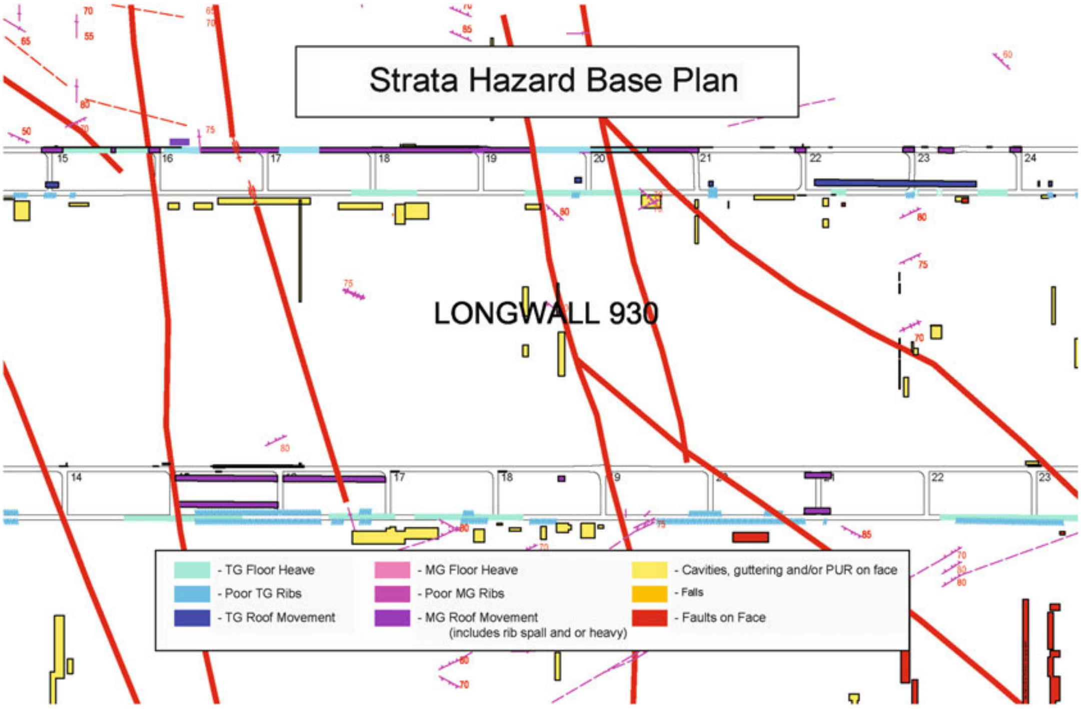

Strata Hazard Base Plan
LONGWALL 930
15
16
17
18
19
20
21
22
23
24
14
17
18
19
22
23
- TG Floor Heave
- Poor TG Ribs
- TG Roof Movement
- MG Floor Heave
- Poor MG Ribs
- MG Roof Movement
(includes rib spall and or heavy)
- Cavities, guttering and/or PUR on face
- Falls
- Faults on Face

**Fig. 12.7** An extract from a Hazard Plan

trigger level is reached. Trigger Action Response Plans can also be used to manage the consequences of a top event should it still occur, in which case they may place a high focus on contingency planning. The concept is illustrated in Table 12.1 using an excerpt from the Trigger Action Response Plan for ground control on a longwall face presented in Appendix 16.

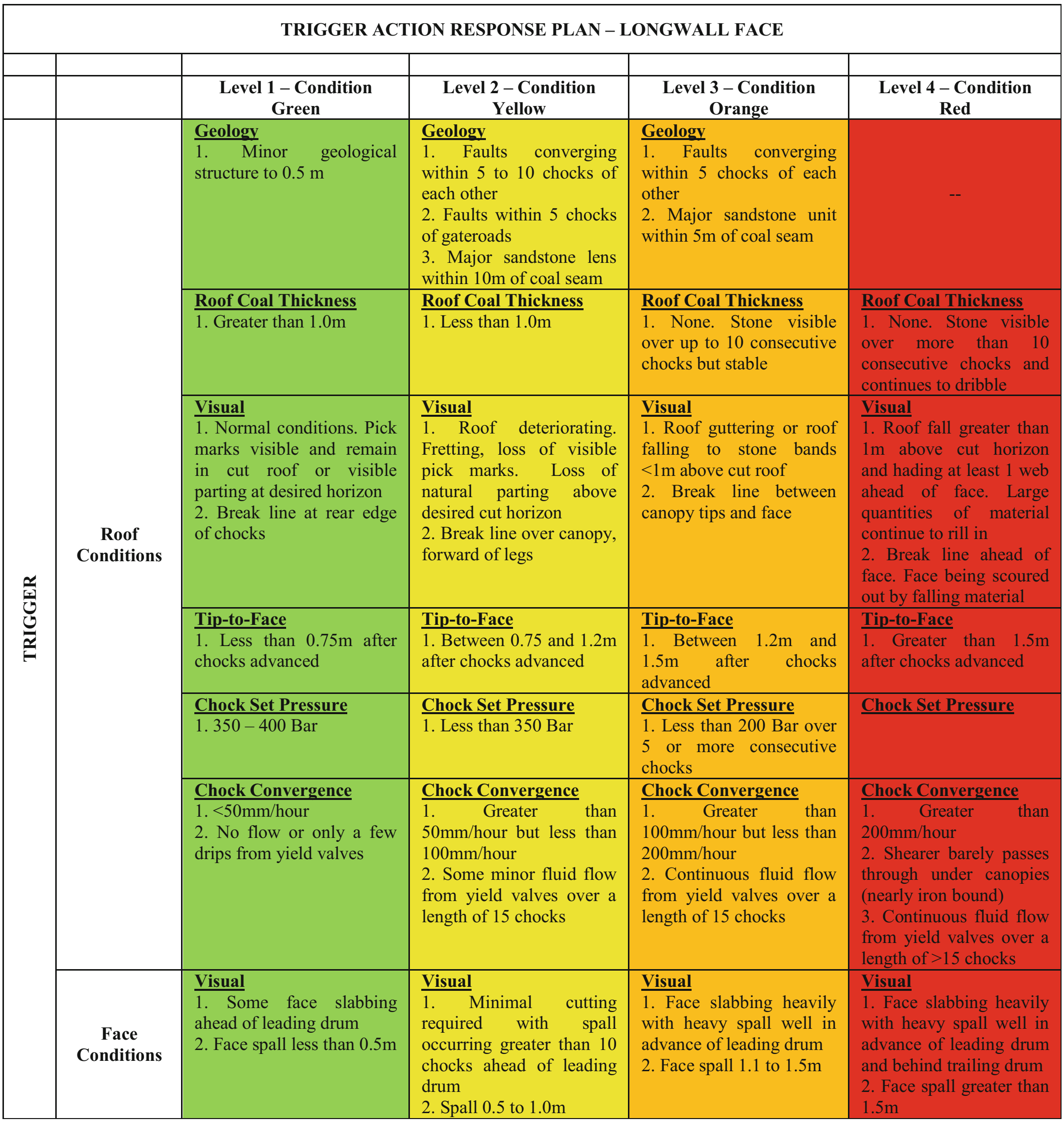
Trigger Action Response Plans are usually tailored to different geotechnical domains or to different operational stages in the mine development, such as outbye, main development, panel development, longwall installation roadway, and secondary extraction. Trigger levels are decided on the basis of the Hazard Plan, strata monitoring experiences, confidence levels in the design process and operational experience, which should include consultation with the workforce.

### 12.6.3 Review

Periodic review of the Ground Control Management Plan, including the underpinning risk assessment, Hazard Plans and Trigger Action Response Plans, is essential to confirm that all critical risks have been identified and that controls remain appropriate and adequate. A number of factors may trigger a review, including:

- the passage of a stipulated period of time, preferably not exceeding 24 months;
- measured rates and amounts of movement that significantly exceed predicted;
- overstressed ground support systems;
- activation of a ‘red’ trigger level in a trigger action response plan;

**Table 12.1** An excerpt from a Trigger Action Response Plan pertaining to ground control on a longwall face, illustrating the concept of staged threshold levels



**TRIGGER ACTION RESPONSE PLAN – LONGWALL FACE**

| | | Level 1 – Condition Green | Level 2 – Condition Yellow | Level 3 – Condition Orange | Level 4 – Condition Red |
|---|---|---|---|---|---|
| TRIGGER | Roof Conditions | **Geology**<br>1. Minor geological structure to 0.5 m | **Geology**<br>1. Faults converging within 5 to 10 chocks of each other<br>2. Faults within 5 chocks of gateroads<br>3. Major sandstone lens within 10m of coal seam | **Geology**<br>1. Faults converging within 5 chocks of each other<br>2. Major sandstone unit within 5m of coal seam | -- |
| | | **Roof Coal Thickness**<br>1. Greater than 1.0m | **Roof Coal Thickness**<br>1. Less than 1.0m | **Roof Coal Thickness**<br>1. None. Stone visible over up to 10 consecutive chocks but stable | **Roof Coal Thickness**<br>1. None. Stone visible over more than 10 consecutive chocks and continues to dribble |
| | | **Visual**<br>1. Normal conditions. Pick marks visible and remain in cut roof or visible parting at desired horizon<br>2. Break line at rear edge of chocks | **Visual**<br>1. Roof deteriorating. Fretting, loss of visible pick marks. Loss of natural parting above desired cut horizon<br>2. Break line over canopy, forward of legs | **Visual**<br>1. Roof guttering or roof falling to stone bands <1m above cut roof<br>2. Break line between canopy tips and face | **Visual**<br>1. Roof fall greater than 1m above cut horizon and hading at least 1 web ahead of face. Large quantities of material continue to rill in<br>2. Break line ahead of face. Face being scoured out by falling material |
| | | **Tip-to-Face**<br>1. Less than 0.75m after chocks advanced | **Tip-to-Face**<br>1. Between 0.75 and 1.2m after chocks advanced | **Tip-to-Face**<br>1. Between 1.2m and 1.5m after chocks advanced | **Tip-to-Face**<br>1. Greater than 1.5m after chocks advanced |
| | | **Chock Set Pressure**<br>1. 350 – 400 Bar | **Chock Set Pressure**<br>1. Less than 350 Bar | **Chock Set Pressure**<br>1. Less than 200 Bar over 5 or more consecutive chocks | **Chock Set Pressure** |
| | | **Chock Convergence**<br>1. <50mm/hour<br>2. No flow or only a few drips from yield valves | **Chock Convergence**<br>1. Greater than 50mm/hour but less than 100mm/hour<br>2. Some minor fluid flow from yield valves over a length of 15 chocks | **Chock Convergence**<br>1. Greater than 100mm/hour but less than 200mm/hour<br>2. Continuous fluid flow from yield valves over a length of 15 chocks | **Chock Convergence**<br>1. Greater than 200mm/hour<br>2. Shearer barely passes through under canopies (nearly iron bound)<br>3. Continuous fluid flow from yield valves over a length of >15 chocks |
| | Face Conditions | **Visual**<br>1. Some face slabbing ahead of leading drum<br>2. Face spall less than 0.5m | **Visual**<br>1. Minimal cutting required with spall occurring greater than 10 chocks ahead of leading drum<br>2. Spall 0.5 to 1.0m | **Visual**<br>1. Face slabbing heavily with heavy spall well in advance of leading drum<br>2. Face spall 1.1 to 1.5m | **Visual**<br>1. Face slabbing heavily with heavy spall well in advance of leading drum and behind trailing drum<br>2. Face spall greater than 1.5m |

- strata failure with the potential to injure people or to adversely impact on maintaining safety in the workplace e.g. roadway convergence impeding travel ways and second egresses; falls of roof or rib; and floor heave impacting on alignment and tracking of conveyor belts;
- a near miss (or near hit) or an incident associated with a loss of ground control.

### 12.6.4 Change Management

A question which should always be at the forefront of management's mind is: What has changed from the last risk assessment that could impact on the effectiveness of the Ground Control Management Plan? One needs to look beyond day-to-day changes in local conditions. For example, changes in regional mine stability may develop incrementally over a considerable period of time, resulting in them going unnoticed or in personnel becoming conditioned to and accepting the changed risk profile without question.

Triggers for risk reassessment include changes associated with:

- mine ownership;
- staff profile and competencies;
- loss of corporate memory;
- mining layouts, dimension, directions, horizons and boundaries;
- mining methods;
- mining technologies;
- updated geological and geotechnical information;
- extensive extraction abutting regional geological features;
- mining operations in adjacent leases;
- mining operations in upper or lower seams;
- progressive deterioration in ground conditions;
- ground conditions falling outside the scope of a trigger action response plan;
- ground support materials and technologies;
- ground support installation techniques and equipment;
- support patterns and timing of installation;
- new or alternative geotechnical assessment techniques and methods.

The risk reassessment process may range from a simple Job Safety Analysis (JSA) undertaken at the workplace by internal personnel, through to a formal risk assessment involving internal and external input. An example of a change management policy is provided in Appendix 17.

### 12.6.5 Other Implementation Considerations

Internal audits, typically conducted annually, and external audits, typically conducted at least every 3 years and more frequently in the event of a major incident, provide a level of assurance that the Ground Control Management Plan has been implemented and is effective and are mechanisms for promoting continuous improvement. However, it is particularly important in the case of ground control that the audits are undertaken against risks as well as standards. That is, in addition to desktop audits of the paper trail that supports the Ground Control Management Plan, evidence should be sought from the work place that the Ground Control Management Plan is effective in managing risk. It is also advisable for the audit to verify that the Ground Control Management Plan is consistent with (maps against) legislative requirements.

A high reliance is sometimes placed on a Ground Control Management Plan fulfilling a quality assurance function. In these cases, regard may need to be had to the requirements of AS/NZS-9001:1994 (1994) for quality assurance in engineering design, development, production, installation and servicing; and to AS/NZS-3905.12:1999 (1999) which provides guidance on satisfying these requirements.

### 12.6.6 Determining Acceptable Levels of Risk

Statistically, there is no such thing as a zero level of risk when it comes to ground control. With the move towards probabilistic base analysis and risk management and the mining industry's aspiration of *zero harm*, the question arises as to what level of design risk is acceptable. The level of risk tolerated by society is not only decreasing but at any point in time varies with the age, number, occupation and country of those at risk. Tolerance figures are defined by so-called F-N plots, where F is an annualised probability that N or more lives will be lost by a defined event. Societal risk is not easily quantified but based on guidance material relating to major facilities (WorkCover Victoria 2000) and to large dams (ANCOLD 2003), typical accepted annualised probability rates for fatalities in Australia as at 2003 were:

- 1 in 1000 years for single fatality events where the victim has control of their environment;

**Table 12.2** Comparison between probabilities assigned to qualitative descriptors of likelihood by the Australian Geomechanics Society (2007) and by Barneich et al. (1996) with those assigned by the general public as determined by Reagan et al. (1989)

| Landslide risk management (Australian Geomechanics Society 2007) | | Dam engineering (Barneich et al. 1996) | General Public (Reagan et al. 1989) | |
|---|---|---|---|---|
| Descriptor | Approximate annual probability (%) | Assigned probability (%) | Median of responses (%) | Descriptor |
| Almost certain | 10 | ~100 | 90 | Almost certain |
| Likely | 1 | 10 | 70 | Likely |
| Possible | 0.1 | 10 | 40 | Possible |
| Unlikely | 0.01 | 1 | 15 | Unlikely |
| Rare | 0.001 | 0.1 | 10 | Very unlikely |
| Barely credible | 0.0001 | 0.01 | 2 | Almost impossible |

- 1 in 10,000 years for single fatality of a member of the public;
- 1 in 100,000 years for a 10 public fatality event;
- 1 in 1,000,000 years for a 100 public fatality event.

However, incidents confirm that mining-related fatalities evoke greater emotion and ongoing press, public outcry, government intervention and judicial inquiry than incidents in the course of many other activities, such as driving or flying. Therefore, it is likely that probability rates for ground control related fatalities need to be considerably lower than the values typically quoted.

Particular care is required when dealing with stability assessment and risk assessment outcomes that are expressed in qualitative terms. Table 12.2 shows two correlations between quantitative and qualitative descriptions of probability that find application in ground engineering, one related to dams (Barneich et al. 1996) and the other to landslides (Australian Geomechanics Society 2007). The probability ranges assigned to the qualitative descriptors by each scheme differ by up to two orders of magnitude. Nevertheless, this can be managed because the actual probabilities have been quantified in each case. The real concern arises when the probabilities assigned to qualitative descriptors are compared to the probabilities that Reagan et al. (1989) determined that the general public associate with these descriptors. For example, the general public perceive that something which is described as *likely* has a 70 % probability of occurring, whilst Barneich et al. (1996) associate the descriptor with a 10 % probability and the Australian Geomechanics Society (2007) with a 1 % probability of occurring.

A further factor which needs to be considered is the design life of the structure. Statistically, the longer the design life, the higher the cumulative probability that an event will occur, with annualised probability of failure determining the time to reach a 100 % probability of failure. An example relating to landslides is presented in Fig. 12.8. This is one motivating reason for increasing the design safety factor, or probability of stability, of coal pillars that comprise main development headings or that are required to provide support to the surface in the very long term.

Ultimately, the owner of the risk needs to decide on the acceptable level of risk for the given circumstances, premised on risk assessment.

### 12.6.7 Reviewing a Risk Assessment

*MDG 1014 – Guide to Reviewing a Mine Risk Assessment* (MDG-1014 1997) notes that an independent review of a risk assessment aims to identify any weaknesses in the process; assist

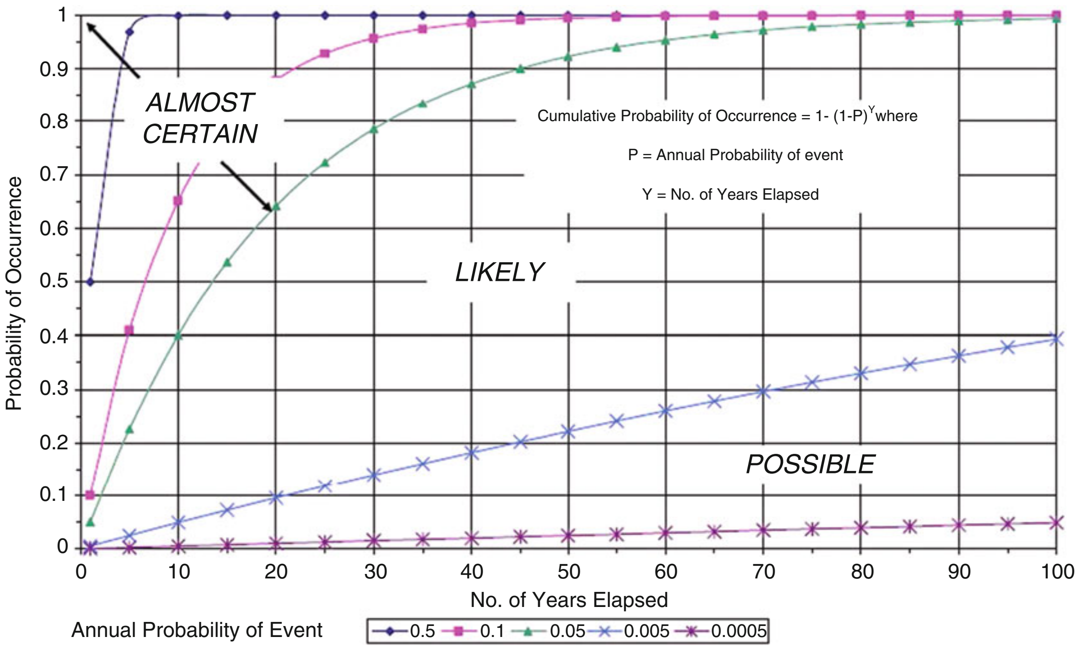


**Fig. 12.8** Indicative probability of occurrence after given number of years, showing corresponding qualitative descriptions of risk adopted in Australian Geomechanics Society's practice notes for landslides (Australian Geomechanics Society 2007)

those responsible for the assessment to remedy the weaknesses in the assessment and in future assessments; and improve performance. This guideline provides a range of information and checklists relevant to developing a robust Ground Control Management Plan premised on risk assessment.

## 12.7 Monitoring

### 12.7.1 Purpose

Monitoring is integral to the effective execution of the management functions of Plan-Do-Check-Act, especially when managing risk. In risk management, monitoring is a control for verifying compliance with design and construction standards; for confirming predicted effects, impacts and consequences; and for detecting deviations from planned performance in a timely manner. It also can give direction to remedial responses and insight into opportunities for continuous improvement. Monitoring is an essential element in ground engineering and management because of the pervasive presence of uncertainty. Trigger Action Response Plans rely on appropriate and effective monitoring.

In the underground coal mining sector, ground monitoring is undertaken to:

- aid in exploration;
- establish benchmark data for environmental approval and licensing purposes;
- determine properties for input into mine design;
- validate mine design;
- validate the quality of ground support hardware;
- validate the quality of ground support installations;
- research the unknown;
- provide timely warning of deviation from predicted ground conditions and design performance, both in the short and long term; and
- identify, quantify and verify mining effects, impacts and consequences.

The need for monitoring is a reflection that the models used to predict rock mass response to different types of mining procedures are based on idealisations, assumptions and simplifications, with monitoring providing feedback to close the design loop (Brady and Brown 2006). This process is shown in Fig. 12.9.

Operating mines constitute laboratories for developing and validating design methodologies utilising monitoring, often supported by back-analysis. As such, monitoring underpins what is often termed the *observation principle* or *observational method* of design, in which models and analysis are continuously reviewed and updated on the basis of field data and performance.

Monitoring is particularly critical in ground engineering for managing the risks associated with unintentional or, in the case of secondary extraction, intentional falls of ground. Ideally, monitoring systems need to be designed and implemented in a manner that provides timely, fail-safe warning of the development of critical ground conditions so that personnel and equipment are not exposed to consequences such as burial, entrapment, windblast, dust, and noxious and flammable atmospheres. Gaps in knowledge and technology currently prevent these monitoring goals from being fully achieved.

### 12.7.2 Monitoring Strategy

A comprehensive monitoring strategy typically needs to specify:

- Monitoring methods
- Monitoring schedule
- Monitoring frequency
- Interpretation techniques
- Triggers
- Responses
- Responsibilities and accountabilities
- Reporting protocols

Appendix 18 presents an example of a Ground Control Monitoring Plan that makes provision for developing these strategies. This plan provides monitoring requirement guidelines for five aspects of ground control management that are common to all underground coal mines, namely:

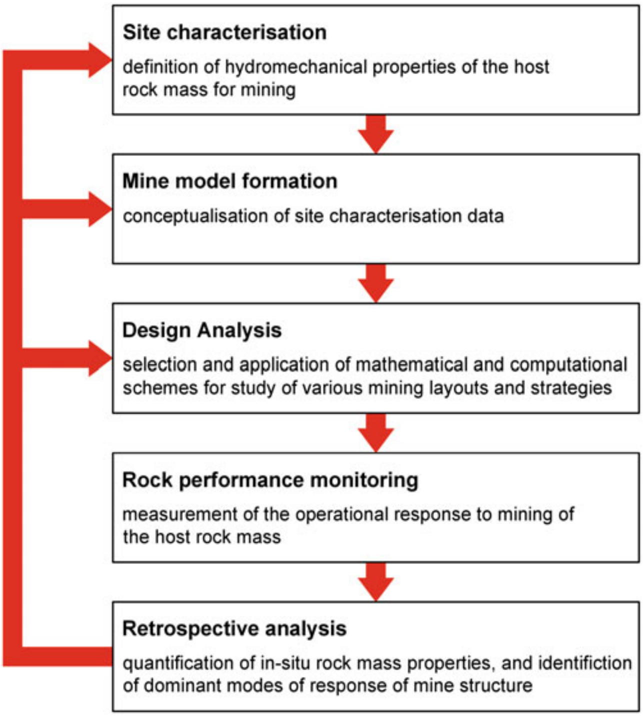

Site characterisation
definition of hydromechanical properties of the host rock mass for mining
Mine model formation
conceptualisation of site characterisation data
Design Analysis
selection and application of mathematical and computational schemes for study of various mining layouts and strategies
Rock performance monitoring
measurement of the operational response to mining of the host rock mass
Retrospective analysis
quantification of in-situ rock mass properties, and identifiction of dominant modes of response of mine structure

**Fig. 12.9** Simplified flow diagram of the ground engineering design process showing the contribution of monitoring (After Brady and Brown 2006)

1. Ground control performance in each mining panel – predicted versus actual
2. Quality control of ground support materials
3. Condition of ground support installation equipment
4. Quality of ground support installation
5. Processing of monitoring data

Points 2 to 4 are fundamental to achieving effective ground control. The considerable effort and expense associated with designing and installing ground support systems can be seriously compromised if the support system elements are not manufactured to specification or not installed correctly. Table 12.3 presents an example of a checklist for verifying the quality of roof bolting components and the standard of their installation. This list, which has been sourced from a different Ground Control Monitoring Plan to that presented in Appendix 18, is not necessarily exhaustive.

### 12.7.3 Sensory Monitoring

When discussing monitoring, there is a tendency to focus on instrumentation. Whilst instrumentation is very important in underground mining, so too is sensory monitoring by the workforce. The workforce are trained and conditioned to 'reading' the state of stability and to detecting changes from the norm through observing, hearing and feeling (ground vibrations/seismic events). They can recognize signs, often subtle, of changing conditions and impending instability that fall outside the knowledge and experience of external personnel and the spatial coverage and detection capacities of instrumentation. These signals can include a change in noise and/or penetration rate of cutting and drilling machinery; a change in the colour of drill cuttings; 'jumps' in a drill string as it crosses parting planes; a change in the colour or fabric of features in the coal face; frequency and intensity

**Table 12.3** Quality control verification checklist for roof bolting

| Support elements | Installation | Compliance with design |
|---|---|---|
| Roof bolts | Correct installation cycle | Spacing |
| Strength of roof bolts | Correct spinning-holding time | Correct bolt |
| Correct length | Correct insertion of resin | Correct resin |
| Correct diameter | Correct drilling | Correct hole size |
| State of corrosion | Correct drill bit size | Correct drill bit size |
| Straightness | Correct rod & hole length | Correct adjustment of roof bolting rigs |
| Resin | Correct flushing | |
| Strength & modulus | Correct roof bolt pattern | |
| Storage | Correct time-to-installation | |
| Type | Correct resin storage | |
| Dimensions | | |
| Borehole | | |
| Diameter & annulus | | |
| Straightness | | |
| Location & inclination | | |
| Length | | |
| Roughness | | |
| Roof bolters | | |
| Torque | | |
| Thrust | | |
| Speed | | |
| Accessories | | |
| Washer strength | | |
| Washer size | | |
| Nut strength | | |
| Thread type | | |

of floor bumping; and subtle signs that are of site-specific significance such as very minor flaking of the immediate roof or ribs.

Furthermore, mine employees can provide valuable insight into failure mechanisms through their descriptions of the timing and manner of the development of instabilities; albeit that they are unlikely to have an appreciation or understanding of the underlying mechanics. They should always be consulted and listened to when assessing strata stability and investigating strata failures.

Observation can constitute the basis for critical decision making in some circumstances, such as the timing of retreat from a lift in pillar extraction. Its effectiveness can be aided by stone dusting the mine workings as soon as practical after excavation. In addition to being a fundamental control for mine explosion, stonedusting improves illumination in the workplace and can provide an excellent reference background for detecting the onset, direction and nature of strata failure. However, if a geological structure is detected, it may be advisable not to stonedust the immediate area so as to avoid obscuring the structure, at least until such times as it has been thoroughly mapped and verified to be in a benign state.

There are a number of circumstances where the sense of hearing can find application as a monitoring tool. These include detecting changes in rock competence when cutting or drilling; sounding the roof to detect parting planes; detecting the presence and rate of convergence based on the 'cracking' of timber props or the 'clacking' of pressure relief valves on longwall hydraulic legs; and predicting the timing and size of impending falls of ground based on the 'goaf talking'. According to MDG-1007 (1996), experience indicates that sounding can detect an open parting up to 1.8 m into the roof in a massive roof setting. However, extreme caution should be exercised with this type of advice. For example, tests by Galvin and McCarthy (1998) associated with a fatal fall of ground determined that, prior to delamination occurring, sounding was only capable of detecting a thin weak mudstone within a massive conglomerate roof strata when the band was within the first 0.8 m of the roof.

As sounding the roof requires the operator to be standing in close proximity to the area being tested for stability, this method should always be underpinned by a safe work procedure premised on risk assessment and training. The success of the method in massive roof strata is dependent on the operator letting go of the steel (striker) as it is about to hit the roof (or bell). In many bedded or laminated immediate roof situations, the roof may only require tapping to determine that it is 'drummy'.

The sensing of movement is a particularly important monitoring aid in total pillar extraction. The detection of 'bumping' of the floor and roof and 'ear popping' (overpressure events) play an important role in monitoring and decision making by operators.

### 12.7.4 Monitoring with Instrumentation

Monitoring instrumentation is based almost exclusively on measuring two basic physical responses, namely displacement and fluid pressure. Brown (1993) emphasised the importance of recognising that the 'measurement' of most other variables of interest, notably forces and stresses, requires the use of a mathematical model and material properties (e.g. elastic constants) to calculate the required values from measured displacements, strains or pressures. As noted by Burland (1967), stress is a philosophical concept - deformation is the physical reality.

The timing and location of instrumentation are fundamental considerations. Rock mass displacement and changes in ground stress and groundwater pressure can be initiated far in advance of the mining face and so, unless instrumentation is also installed well ahead of mining in virgin conditions, absolute values of these changes may not be able to be determined.

Installation location can affect the exposure of instrumentation to shearing on bedding planes. Instruments installed on the flanks of an excavation are prone to becoming unserviceable due to damage caused by shearing on parting planes.

Measurement of absolute displacement requires a stable reference point. This can be difficult to achieve in an underground environment. Firstly, unless the instrument is installed well ahead of mining, it is inevitable that some movement will already have occurred prior to installation. Secondly, mines are dynamic environments and so one may have to go far afield to find a reference point outside the ongoing influence of mining. This introduces the opportunity for error in relating measurements back to this stable point. Fortunately, as a high level of precision is not required for most purposes, this second limitation can usually be managed without introducing significant error by assuming a fixed reference point in reasonable proximity to the measurement site.

A measurement of normal strain can be obtained by assuming that strain is uniformly distributed between two points. The closer the points, the more likely that the calculated uniform strain reflects the true strain. Particular care is required in situations where structural stability is critically dependent on limiting strain, such as mining beneath water bodies or surface structures. In these circumstances, a calculated uniform strain of, say 3 mm/m over a 20 m bay length (survey peg interval), could in reality correspond to a 60 mm wide crack somewhere in that bay length, giving rise to very different consequences and, hence, risk profiles. This is an example of a fundamental risk management aspect of monitoring that must always be questioned, namely:

> *Are you actually measuring what you think you are?*

More detailed information on instrumentation to support monitoring programs can be sourced from specialist instrumentation providers and a range of publications such as Brady and Brown (2006), Peng (2008), Pidgeon et al. (2011) and Mills (2011).

## 12.7.5 Displacement Monitoring Instrumentation

### 12.7.5.1 Borescope

A borescope, or stratascope, is a flexible periscope that permits visual and photographic observations in a borehole. The trunk of the device comprises bundles of fibre optic cables. One bundle is used to transmit the image of the borehole walls to the eyepiece and the others are used to transmit light to the back of the instrument. The lighting source is a cap lamp or other external battery powered torch.

The instrument can be used to detect geological composition, fractures, bed separation and shearing, typically over a distance of up to 6 m. Advances in lighting sources and digital technology are resulting in borescopes being replaced by down-the-hole digital camera technology in many applications. An advantage of this technology is that the borehole conditions can be observed over distances in excess of 100 m.

### 12.7.5.2 Convergence Pole

A convergence pole comprises an inner and outer tube that are pushed apart by an internal spring, with a device such as a tape measure or potentiometer mounted on the tubes to measure their relative displacement. They find most application in measuring closure between the roof and the floor of mine workings. A limitation with these types of devices is that the respective contribution of roof and floor movement cannot be determined. This can be resolved in some situations by also monitoring the intercept of the convergence pole and a string line strung between the ribs. The use of electrical transducers such as potentiometers and linear variable displacement transducers (LVDTs) facilitates reading the instruments remotely and/or continuously with a data logger.

### 12.7.5.3 Extensometer

An extensometer is a device used to measure the displacement of a point relative to a datum point. Most extensometers are of a multipoint variety, comprising a series of anchored points along the length of a borehole. In surface installations, an example of which is shown in Fig. 12.10, anchor displacement is measured relative to the collar assembly at the mouth of the borehole. If measurement of absolute displacements is required and the surface is subject to movement such as caused by mine subsidence, the elevation of the collar assembly has to be monitored by precise levelling back to a known fixed datum.

The determination of absolute displacements in underground extensometer installations is usually based on assuming that the deepest anchor is unaffected by mining. An example of a simple two anchor mechanical extensometer that is often used to monitor the roof bolted horizon is shown in Fig. 12.11. The rear anchor is located above the expected height of delamination and the lower anchor just below the top of the bolted horizon, thereby providing measurements of dilation within the bolted horizon and total dilation.

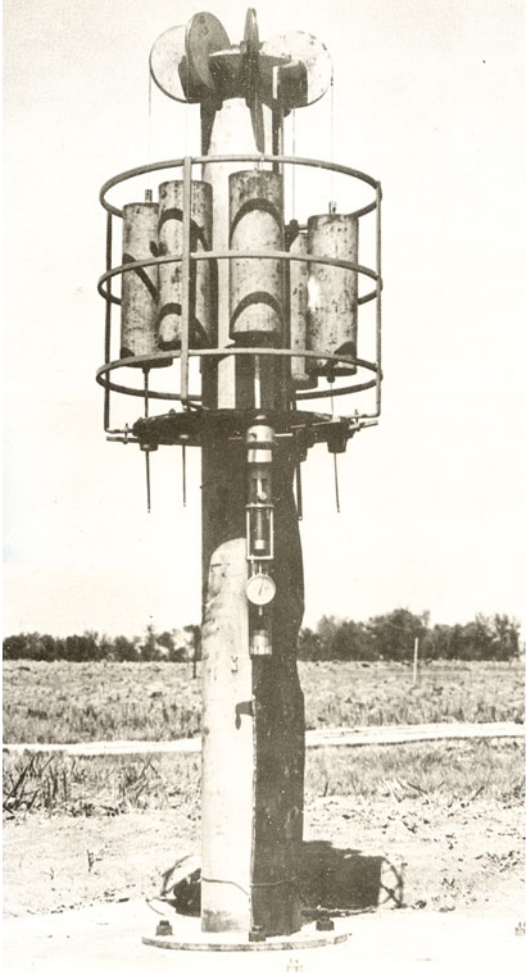


**Fig. 12.10** A six point surface to seam extensometer installation capable of being read to 0.01 mm by means of a dial gauge and pretensioning procedure, used in confirming the applicability of elastic theory to underground coal mining environments (After Oravecz 1973)

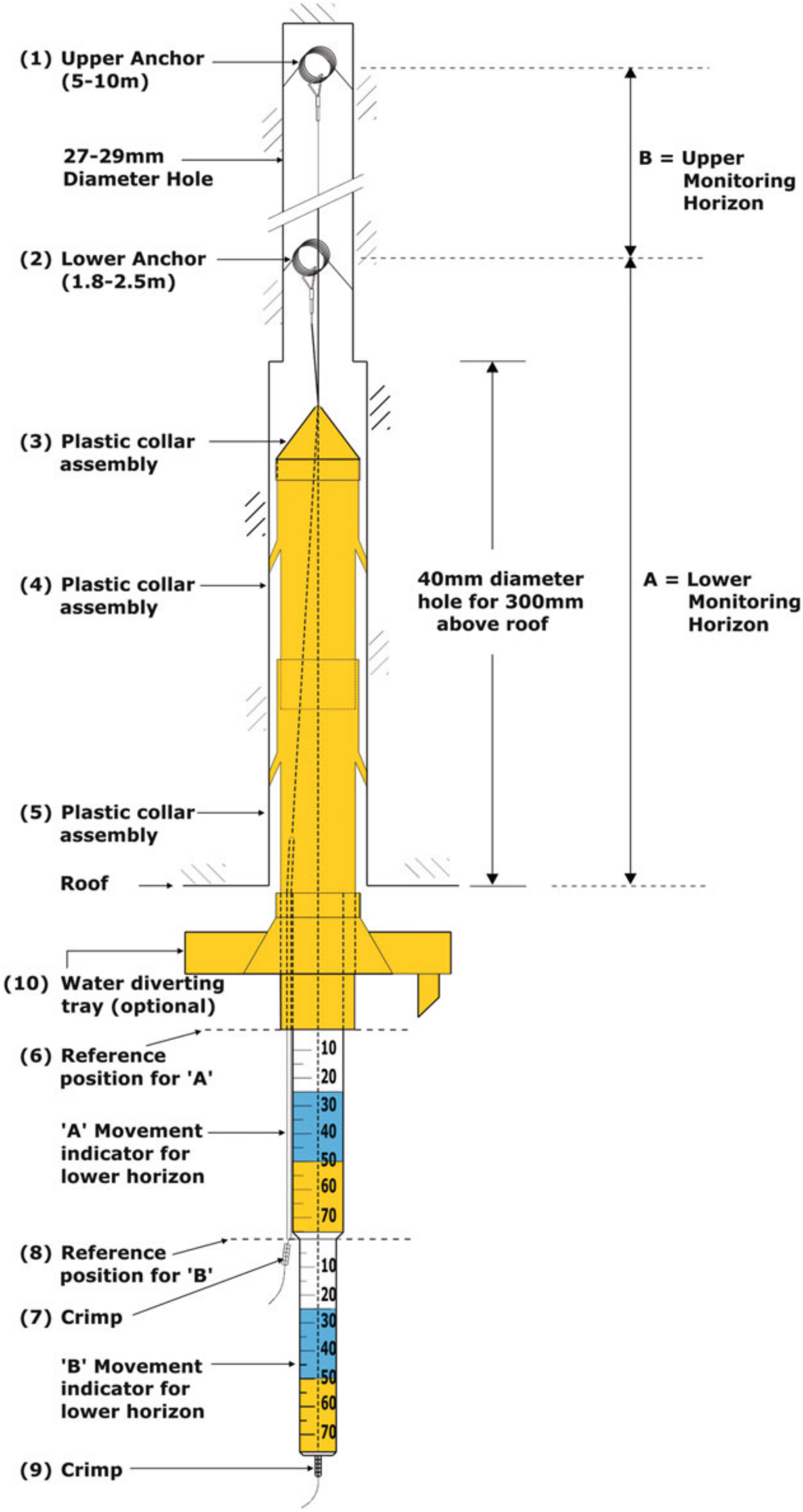
The number of anchors within a single hole is limited by the need to avoid interference between the wires. The most common means of monitoring the displacement of an anchor is by way of a wire connecting the anchor to some measuring device at the mouth of the borehole. The measuring device may be as simple as a section of tape measure attached permanently to the end of the wire (Fig. 12.11), a more precise dial gauge (Fig. 12.10), or an electric transducer. All these types of installations have the advantage of enabling displacements to be computed at the installation site at the time of reading the extensometer.

A sonic probe is a more sophisticated form of extensometer that typically permits displacement to be monitored accurately at up to 20 horizons in a borehole. The installation comprises a series of magnetic rings that are pushed up a hole to target horizons and held in place by friction. A flexible tube passes through the rings and serves as a guide for both installing the magnets and for the sonic probe that is inserted up the hole to measure the relative location of each magnet. The sonic probe is a magnetostrictive sensor. When an electrical pulse is sent down the probe, it creates a magnetic field which interacts with the magnetic field of each ring anchor. The change induced in the magnetic field of the probe generates a torsional stress pulse, or sonic pulse, that travels along the probe. The arrival time of this pulse is used to determine the position of each magnetic anchor to an accuracy of 0.1 mm or better, relative to a reference magnet near the mouth of the borehole.

Limitations associated with this measuring technique include that the reading probe is of limited length; the probe is susceptible to damage in an underground environment; measurements cannot be processed at the installation site; and calibration is unique to each probe. Hence, reading continuity is lost if a replacement probe is not calibrated beforehand. These factors result in this type of extensometer being used mainly for research purposes. It is advisable to have access to a pre-calibrated backup probe.

Irrespective of the design of an extensometer, the relative displacement between extensometer

**Fig. 12.11** A two point anchor extensometer installed in a blind borehole


(1) Upper Anchor (5-10m)
27-29mm Diameter Hole
(2) Lower Anchor (1.8-2.5m)
(3) Plastic collar assembly
(4) Plastic collar assembly
(5) Plastic collar assembly
Roof
(10) Water diverting tray (optional)
(6) Reference position for 'A'
'A' Movement indicator for lower horizon
(8) Reference position for 'B'
(7) Crimp
'B' Movement indicator for lower horizon
(9) Crimp
B = Upper Monitoring Horizon
A = Lower Monitoring Horizon
40mm diameter hole for 300mm above roof
10
20
30
40
50
60
70
10
20
30
40
50
60
70

anchors can be used to generate strain profiles provided that the individual anchor locations are known. Plots of absolute and relative displacement and strain profiles enable the progress and nature of deformation (tensile or compressive) to be monitored and the location of bed separation horizons and crush zones to be identified (Fig. 12.12).

Extensometers find extensive application as monitoring devices in underground coal mining, especially in longwall gateroads where they may be installed at intervals as close as every 25 m to monitor the effectiveness of reinforcement tendons. Extensometers consisting of two anchors are most often employed for this purpose.

The following aspects are particularly important in relation to the installation and use of extensometers, especially given their extensive use and the high reliance placed on them:

- Consideration must always be given to how much movement may have occurred prior to the installation of the instrumentation. If an absolute value of displacement is set as a trigger level when the amount of displacement that may have occurred prior to installing the extensometer is not known with any degree of certainty, then the trigger level needs to be appropriately conservative.
- Earlier warning of a developing instability can often be obtained by plotting the rate of change of displacement, being velocity, and the rate of change of velocity, being acceleration. Figure 12.13 has been developed to illustrate this important point. The example shows that an increase in the rate of strata movement can be detected one week earlier in the velocity and acceleration plots than in the displacement plot. This is one reason why consideration should be given to also setting trigger levels for velocity and/or acceleration. A range of experience attests to velocity being a particularly valuable indicator of impending instability in pillar extraction (for example, reference Maleki 1992 and Marshall Miller and Associates 2006).
- A second reason for including velocity and/or acceleration as a trigger level is that these measures of change are absolute values that are independent of the amount of displacement that may have occurred prior to the commencement of monitoring.
- Unless an extensometer is installed exactly along a plane of symmetry, a component of the measured displacement may comprise

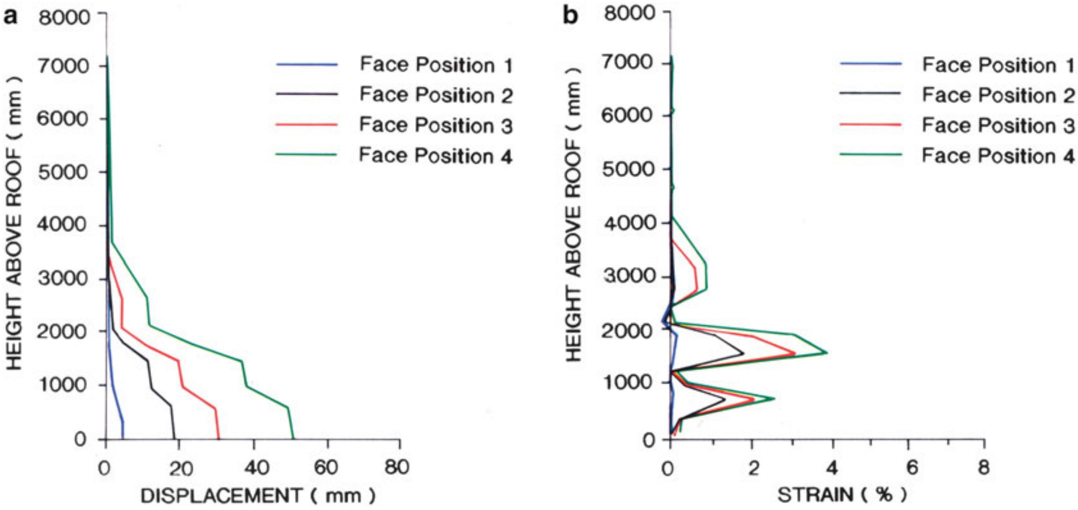


**Fig. 12.12** Output of a multipoint extensometer showing (**a**) the development of displacement and (**b**) strain along the length of a borehole in the immediate roof, with strain spikes indicating the locations of partings

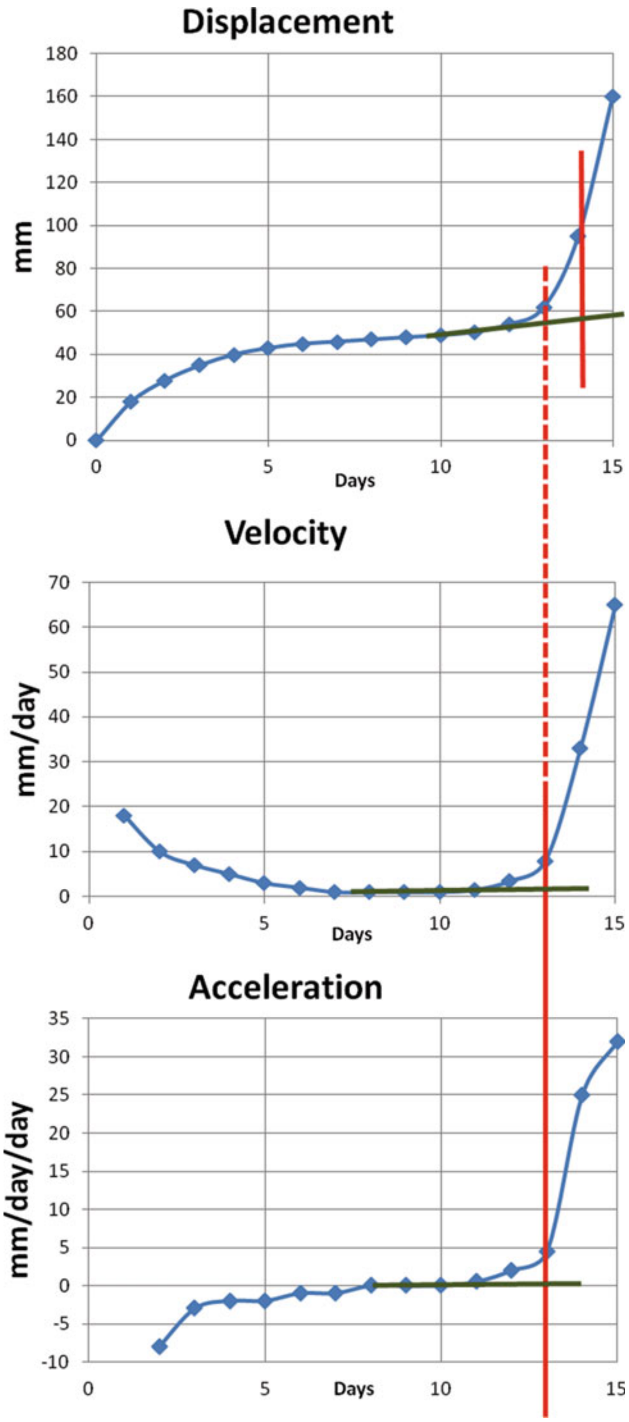


**Fig. 12.13** Extensometer measurements plotted as displacement-time, velocity-time and acceleration-time to demonstrate how velocity and acceleration plots can give earlier warning than displacement plots of a developing unstable situation

shear displacement, as evident in Figs. 6.19 and 6.20. Precise levelling of the collar of the installation and the use of devices such as shear strips can aid in resolving measured displacement into its axial and shear components but this can be difficult and is rarely possible or sufficiently reliable to warrant the effort.

- For the preceding reasons, extensometers that are utilised to monitor immediate roof stability in a roadway are usually most effective and reliable if installed towards the centre of the roadway, at the face.
- Spurious readings can be obtained unknowingly in the case of wire based extensometers if the wires have become tangled during installation. This situation is both more prone to occur and more difficult to detect in long boreholes.
- Anchors are prone to slip. This can create false alarms and false interpretations of ground behaviour. Usually, the situation can be detected by plotting strain.
- False security can be provided in situations where dilation extends beyond an anchor that is being relied upon as a fixed datum. This situation may develop over time or in a step manner and not be detected unless measurements are routinely plotted and interrogated.
- If an anchor of a multipoint surface-to-seam extensometer is placed close to the roof horizon or in the seam or floor horizon of an active mining seam, the entire installation is prone to be lost if a wire becomes tangled in the cutter head of the mining machine.
- If multiple adjacent extensometer installations are employed due to limitations on the number of anchors able to be installed in a single hole, and the deepest anchor is being relied upon as the fixed datum point, this anchor needs to be placed at the same depth as the shallowest anchor in the next deepest hole in order to be able to reference all readings to the same datum point.
- Anchors, anchor wires and the corridors through which they pass are prone to corrode with the passage of time, resulting in additional interference in the borehole and loss of serviceability.

### 12.7.6 Stress Monitoring Instrumentation

Enever (2008) reviewed stress measurement techniques that, by the year 2000, had proven generally tractable enough to be considered acceptable by practitioners for non-research applications and classified them broadly into four categories. These categories have been adopted in this text, being:

- Measurement of strains/deformations associated with total or partial stress of a region of the rock mass at an accessible free surface.
- Measurement of strains/deformations associated with total or partial stress relief of a region of the rock mass adjacent to a borehole drilled from the surface or from an excavation.
- Measurement of fluid pressure in induced or naturally existing fractures in the rock mass.
- Measurement of the profile of boreholes deformed by their existence within a stress field.

#### 12.7.6.1 Free Surface Techniques

The formation of an excavation in a rock mass induces changes in stress profiles and magnitudes around the boundaries of the excavation (as discussed in Chap. 3). Hence, techniques which endeavour to measure stress at a free (unconfined) surface in an underground environment are limited to measuring resultant (total) stresses or, if conducted over a period of time, mining-induced stresses. In practice, however, even this can be dubious since the process of installing the stress measurement instrumentation causes a further change in the stress field that one is trying to measure.

Free surface techniques are based on measuring changes in hydraulic pressure in a flat, thin walled hydraulic jack, known as a **flatjack**, that is grouted into a slot cut perpendicularly into the wall of the free surface. The flatjack is operated in a closed circuit mode and so displacement across a slot is reflected in a change in hydraulic pressure within the vessel. The relationship between displacement and pressure is known by having previously calibrated the flatjack in the laboratory.

Prior to cutting the installation slot, a number of pins are installed on either side of the slot site and the distance between these pins is accurately measured. The formation of the slot causes a partial unloading of the rock mass around the slot, resulting in a degree of slot closure that is determined by measuring the new distances between pins. Once the flatjack is installed, it is pressurised so as to reload the rock mass and return the pins to their original position. The pressure required to achieve this is equated to be the stress acting normal to the plane of the slot prior to its formation.

In order to determine the complete stress tensor, flatjack measurements need to be made in three mutually perpendicular directions, provided that the slots can be orientated perpendicular to the principal stress directions. If the principal stress directions are unknown, then a minimum of six mutually inclined slots is required. This makes no provision for redundancy, which is an important consideration when using flatjacks as they are prone to leak. In any event, the reliability of this technique for determining the complete stress tensor in an underground setting is low since, inevitably, the complete suite of stress measurements cannot be undertaken at the one site. For example, if two orthogonal slots are cut in the ribsides of an excavation, the third orthogonal slot has to be cut in the roof or floor horizon and hence, in a different stress environment.

Advantages of stress measurement methods based on flatjacks include:

- they do not require knowledge of the elastic properties of the rock;
- they are not impacted to any significant extent by small non-uniformities in the rock; and
- the equipment is simple and inexpensive.

These advantages tend to be outweighed by disadvantages, which include:

- the stress field close to the surface of an excavation may be determined by localised features and may not reflect that based on theoretical principles;
- the stress field about an excavation is unlikely to be homogenous, thus limiting the scope of the technique to reliably determine the complete stress tensor;
- results can be affected by rock creep during the process of excavating a slot;
- if the rock around a slot fractures, the method is no longer applicable as elastic stress distribution no longer applies;
- laboratory calibration conditions may not be adequately representative of field conditions, especially in respect of the surface area of each flatjack face that acts on the rock mass;
- there is no capacity for inbuilt redundancy; and
- the devices are susceptible to hydraulic leaks.

These factors in conjunction with advances in other stress measuring techniques have caused a significant decline in the use of free surface stress measuring techniques. However, there is a range of applications in underground mining where they are still useful. These relate primarily to situations where stress only needs to be determined in one dimension such as beneath a goaf, around the perimeter of backfilled workings and within backfill material.

#### 12.7.6.2 Borehole Stress Relief Techniques

Techniques which endeavour to measure in-situ stress down a borehole are characterised by overcoring some form of device that measures displacements induced by the overcoring. Estimated material properties and mathematical routines based on assumed rock behaviour are then used to back-calculate the stress changes required to induce the measured amount of movements. The same devices may also not be overcored and, instead, left in situ to monitor mining-induced stress changes.

Borehole stress relief devices measure displacements utilising either vibrating wire transducers or strain gauges. A vibrating wire transducer comprises a hollow steel cylinder that has a highly tensioned wire strung diametrically across the cylinder walls. The vibration frequency of the wire is calibrated in the laboratory to the stress acting on the cylinder parallel to the direction of the wire. The cylinder is wedged in a position that aims to have the wire orientated parallel to the stress change that one is endeavouring to measure. A change in this stress causes a change in the diameter of the cylinder and, thus, a change in the vibration frequency of the wire.

A strain gauge comprises a thin conductive wire that is arranged in a series of long, closely spaced loops mounted on an insulated backing. The pattern of the wires, results in a small change in the length of the strain gauge parallel to the direction of the wires, inducing a large change in the resistance of the wire circuit. Change in resistance is correlated to displacement, or the load required to cause the displacement, by a previously determined calibration factor known as the *gauge factor*. A strain gauge is bonded to the surface undergoing displacement, with its range determined by the elastic limit of the wire loops. Stress can be determined in two dimensions using a strain gauge rosette comprising three strain gauges orientated in three different directions, these usually being at 0°, 45° and 90°. Displacements can be determined in three dimensions by using orthogonal arrays of strain gauge rosettes. Figure 12.14 shows a strain gauge rosette in a cut-away (half) longitudinal section of a three-dimensional stress measuring cell.

Borehole stress relief techniques can be subdivided into those that involve overcoring a device installed on the face of the end of a borehole and those that involve drilling a larger borehole around a borehole that already has a device attached to its walls. In the former case, the end of a borehole is ground to a smooth surface and a strain gauged measuring cell is bonded to this surface. The borehole is then extended by overcoring the device. The CSIR ‘doorstopper’ is an example of this type of device. The main disadvantage with these types of devices is that they can only measure strain and, therefore, stress in two dimensions.

Borehole overcoring techniques involve boring a small diameter pilot hole in the face of a larger diameter borehole and securing an instrument containing displacement measuring sensors to the wall of the pilot hole (Fig. 12.14). The pilot hole is then overcored using a thin walled core barrel of the same diameter as the outer borehole. This is designed to produce a stress relieved, thick-walled cylinder of rock containing the instrument. The cylinder is tested in the laboratory to determine the elastic properties of the rock in order to convert measured strains to stresses. Displacements can be measured in three dimensions by strain gauges that are bonded either directly to the borehole walls or embedded within an inclusion that is bonded to the borehole walls. The CSIR triaxial strain cell, the CSIRO Hollow Inclusion cell and the ANZI cell are examples of these devices.

No borehole stress relief measurement device finds universal application, with each having its advantages and disadvantages. In general, these devices are not suitable for use in materials with a high density of fracturing, such as coal. This is one reason why most stress measurements undertaken in underground coal mines are made outside of the coal seam, in a more competent roof or floor stratum and reported relative to a reference rock modulus. This modulus is often chosen to be 10 MPa. Notwithstanding this, the ANZI cell has met with some success in coal environments and offers a number of advantages over many other devices.

The ANZI (Australia New Zealand Inflatable) stress cell was developed in the 1980s and known initially as the ANZSI (Auckland New Zealand Soft Inclusion) cell (Mills and Pender 1986). It comprises a soft, inflatable, cylindrical, polyurethane membrane with 18 strain gauges mounted flush on its outer surface. The surface of the membrane, including the strain gauges, is coated with a low slump epoxy cement, inserted into the pilot borehole and pneumatically inflated at the target horizon. After the bonding agent has cured, the cell is pressure tested to confirm the correct operation of all the strain gauges and to provide a measurement of the in situ modulus of the undisturbed rock. The cell can then be left in situ to monitor mining-induced stress changes or overcored for the purpose of calculating in situ stresses Fig. 12.15.

Stress relief can be monitored throughout overcoring via a cable that passes down the

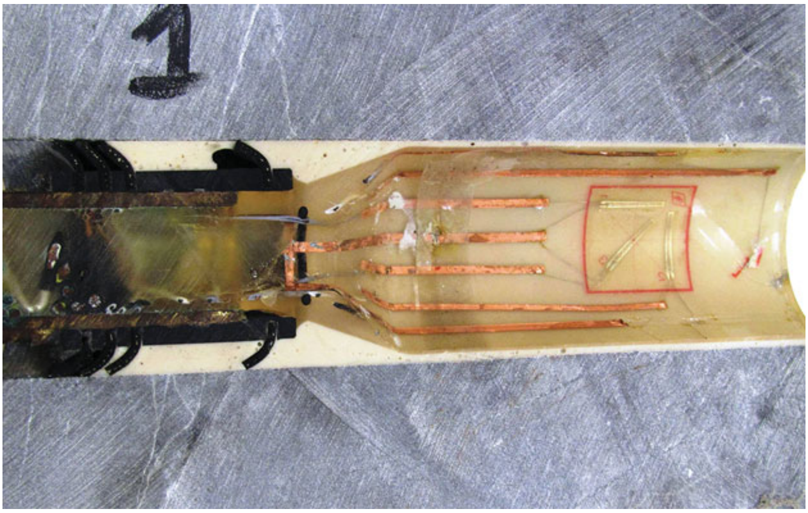


**Fig. 12.14** A strain gauge rosette shown bonded to the surrounding rock mass in a cut-away (split half) longitudinal section of a CSIRO Hollow Inclusion stress measuring cell recovered by overcoring

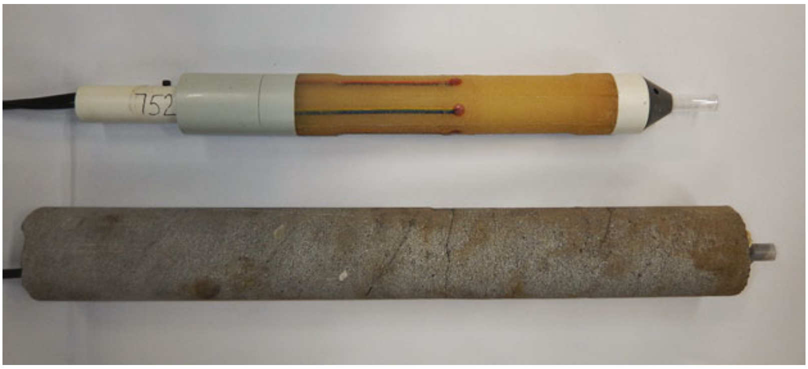


**Fig. 12.15** An ANZI borehole stress measurement cell in both an uninstalled state and embedded in an overcore

centre of the drill string and out through the water swivel. The 18 strain gauges contained within 6 rosettes distributed at 60° intervals around the circumference of the device give 12 degrees of redundancy; permit internal cross checking between independent strain gauges; and are sufficient to allow two completely independent stress determinations from each test. The hollow construction of the instrument also allows up to three stress measurement cells to be installed in the pilot hole at different depths and overcored in the same operation. After the core is recovered, it is subjected to biaxial testing in the laboratory to determine its elastic modulus and Poisson's ratio. The device and its applications are described in more detail by Mills et al. (2012, 2015).

Irrespective of the type of device utilised, a range of complexities are associated with the use of displacement based stress measurement instruments, even in homogenous and undisturbed geological settings. Fundamentally, the process of installing the device can change the existing stress regime and affect future changes in the stress regime that one is endeavouring to measure. The devices can be particularly sensitive to installation procedure, installation horizon, orientation relative to bedding, and micro fracturing. The laboratory determination of material properties is not necessarily straightforward and accurate. Numerical modelling by Corthesy and Leite (2008) has suggested, for example, that as overcore drilling progresses, tensile stress may reach a threshold value, resulting in tensile yielding propagating through to the centre of the core as the drill bit passes.

Brown (2012a) cautioned that the stress path followed by the recovered core and by the surrounding rock mass during the drilling and core recovery process can be expected to have a significant influence on the damage suffered by the core. Brown also noted that, quite often, attempts to measure the stress tensor at a number of points with a view to establishing the pre-mining stress field yield unsatisfactory results. This is not only because of measurement error, but also because of the inherent variability of the stress field resulting from, inter alia, variations in rock types and their mechanical properties and from the influence of structural features.

In summary, stress measurement within underground coal mines is basically confined to borehole stress relief techniques. These are costly, difficult to perform because of the nature of the rock environment and often yield unreliable results. They require specialist equipment and considerable experience. For these reasons, such measurements fall outside the activities normally performed by mine site personnel and should be conducted by specialist organisations.

#### 12.7.6.3 Hydrofracturing Techniques

Hydrofracturing, or hydraulic fracturing, involves sealing off a section of a borehole and pressurising this section with an hydraulic fluid until fluid is lost through either a pre-existing fracture or through a new fracture developed as a result of fluid pressure exceeding the tensile strength of the rock. When a new fracture is formed, pumping is stopped and the fluid in the fracture is allowed to dissipate until the fracture closes. The pressure at which this occurs is indicative of the minor principal stress. The fracture is then reopened and closed several times in a process which enables the major principal stresses to be estimated, provided that the borehole lies in the axis of a principal stress and the rock is not highly inelastic. Techniques such as impression packers, acoustic scanning and down-the-hole photography are used to determine the direction of a new fracture and, hence, the orientation of the principal stresses.

Serious limitations are associated with the technique when the test borehole is not orientated in the direction of a principal stress and/or when the rock mass is already fractured. The latter is mostly the case in coal seams due to the presence of cleating, in which case hydrofracturing only enables the stress magnitude normal to a fracture to be determined. The situation is complicated further when there are two orthogonal sets of cleat.

Care is required to ensure that fracturing is not induced by the packers used to isolate the test section within a borehole, since these must always be at a greater pressure that the hydrofracturing fluid in order to prevent this fluid from leaking past the packers. Packer-induced fractures have the same orientation as fluid-induced fractures.

#### 12.7.6.4 Borehole Profile Techniques

The deviation of an in situ stress field around a circular borehole may lead to a detectable distortion in the borehole profile. Borehole profile techniques rely on relating the nature and extent of profile distortion to the in situ stress field. The direction of the stress field responsible for the distortion becomes obvious if it results in break-out or ‘dog earing’, causing the borehole profile to become elliptical. The detection of less subtle distortions is a function of, firstly, the degree to which the drilling process results in the initial shape of the borehole being smooth and circular; and, secondly, the sensitivity of the distortion measuring equipment.

Borehole deformation measurements can be conducted in both completed boreholes and in pilot holes drilled intermittently as a borehole is developed. Acoustic scanners and caliper tools are used to measure distortion in completed boreholes. In general, these only detect distortion above a threshold level and do not enable the magnitude of the stress field to be determined. More precise instrumentation such as the USBM deformation gauge and the Sigra In Situ Testing (IST) tool is deployed in pilot holes. These are biaxial devices that use a series of pins acting against the pilot hole wall to measure changes in hole diameter during overcoring. This information is processed to produce predictions of both stress magnitude and stress direction. In all cases, careful consideration needs to be given to the accuracy of stress magnitude predictions.

## 12.7.7 Other Instrumentation

### 12.7.7.1 Load and Pressure Monitoring

Load monitoring is usually conducted for the purposes of determining load distribution, peak support capacity and reserve support capacity. It finds most application in the case of standing support systems (props, chocks), tendon systems, mobile roof supports and longwall powered supports. Load cells are used in situations where the total load is concentrated at a single point such as at the top or bottom of a timber prop or at the nut of an end anchored tendon. Common types of construction include washers and disc springs that deform in a characteristic manner at a known load; and strain gauged, thick wall, tubular or solid cylindrical blocks that have been calibrated in a laboratory.

In the case of a fully encapsulated solid bar tendon, the load (or strictly speaking, the force) in the bar can vary along its length as illustrated, for example, in Figs. 6.38 and 6.40. Furthermore, this load can be generated by axial strain or bending strain or a combination of both. A modified bar, or bolt, fitted with strain gauges is required to measure load magnitudes and distributions in these circumstances. Two slots are milled on opposite sides of the bar and fitted with pairs of diametrically opposed strain gauges in order to enable resolution of axially induced load and bending induced load. The accuracy of this technique relies on pre-empting the direction of bending and orientating the rock bolt accordingly. When tendons function to resist bending of the reinforced strata, an array of instrumented bolts is required in order to account for the variation in shear stress across the width of an excavation (refer to Fig. 6.20). The complexities involved in undertaking force measurements in fully encapsulated bolts result in these measurements not being undertaken routinely but rather as part of targeted support investigation programs.

Load determination in the case of hydraulically powered mobile roof supports (MRS) and longwall powered supports is based on summing the outcomes of multiplying the fluid pressure in each leg of a unit by the leg's smallest internal cross-sectional area. In the case of longwall mining, real time monitoring of hydraulic leg pressure trends and the yield behaviour of each support, such as shown in Fig. 12.16, provides detailed information on the state of the hydraulic systems and insight into the state of face stability. In particular, it has the potential to provide early warning of the development of periodic weighting and roof cavities. The monitoring outputs are processed for this purpose using algorithms that have regard to factors such as loading rates, yield frequencies, time weighted average pressures and the number and relative location of affected powered supports at any point in time. Hoyer (2011) and Wiklund et al. (2011) provide more detailed descriptions of these monitoring systems.

All longwall mines in Australia are equipped with real time leg pressure monitoring to aid in identifying and better understanding roof control behaviour and in predicting the onset of poor conditions. However, once legs reach yield, little additional information can be gained about powered support behaviour from monitoring leg pressures. This is becoming more prevalent with the trend towards higher set pressures that results in legs reaching yield pressure sooner. An important monitoring parameter is convergence as this has implications for both ground control and equipment clearance. Since convergence results in a change in the slope of some components of a powered support, such as the caving shield and lemniscate linkages, there is potential to monitor convergence in real time by fitting tilt meters to these types of components. This is an emerging technology (ACARP 2014).

### 12.7.7.2 Seismicity

Seismic monitoring is a global rock mass investigation and monitoring technique that involves the simultaneous measurement of ground vibrations, or seismic waves, at a spread of geographical locations using an array of vibration sensors such as geophones and accelerometers (Fig. 12.17). The seismic waves can be generated naturally or artificially. Natural wave generation

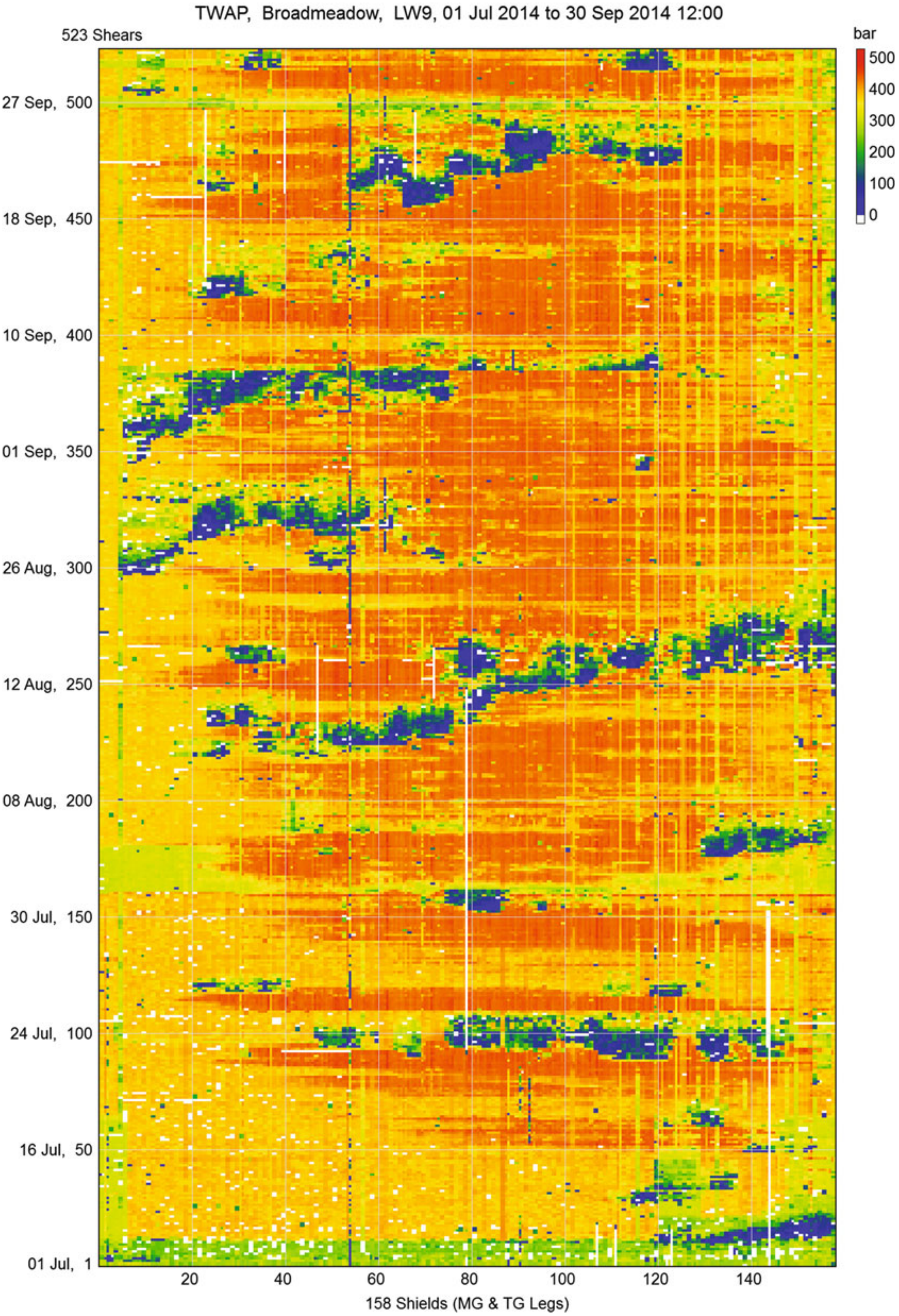


**Fig. 12.16** Real time monitoring extending over 3 months of leg pressure across a longwall face

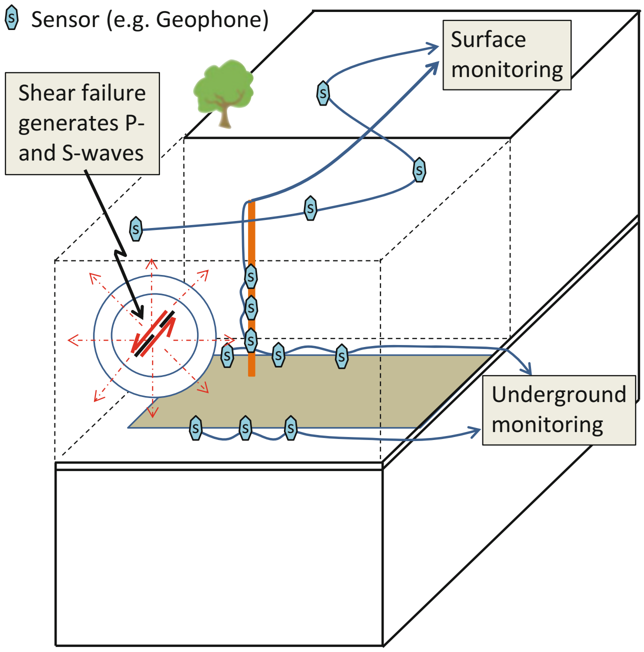


**Fig. 12.17** A schematic layout of a seismic monitoring system

is associated with the development of fractures and failures surfaces within the rock mass and covers a wide spectrum depending on the nature of the source (e.g. micro-fracture development, caving, pillar system collapse, earthquakes). Artificial generation of seismic waves is required with utilising seismic monitoring to investigate the nature of the rock mass. Energy sources include explosive charges and devices that thump the ground.

The geophones detect the arrival times, amplitudes and duration of the seismic waves. The information provided by an array of geophones is processed using algorithms that, in the case of monitoring for natural deformation events, determine the location (epicentre) and magnitude of a seismic event and the type of fracturing associated with it (intact shear, intact tensile, bedding plane shear, reactivation of a shear plane etc). When seismic monitoring is utilised for investigative purposes, processing algorithms are designed to determine rock mass fabric and structure on the basis of the reflection patterns of the seismic waves.

Microseismic monitoring is a variant of seismic monitoring that is concerned with detecting very low energy release events associated with the development of micro-fractures that ultimately lead to rock failure. It is often employed in coal mines that work under strong massive roof strata that do not cave readily, resulting in periodic weighting and windblast conditions presenting an elevated risk. Microseismic monitoring can provide valuable information concerning strata behaviour in such conditions. These matters are discussed in Sects. 3.3 and 11.2. The reader is referred to the wide range of literature available on seismic monitoring, including Kelly et al. (1996), Hatherly and Luo (1999), Hatherly et al. (2003) and Shen et al. (2013).

#### 12.7.7.3 Shear Movement Detection

A shear strip comprises a flat stainless steel bar with strain gauges bonded at closely spaced intervals, typically 5 mm, on opposite sides of the bar. The bar is housed in a plastic tube that is grouted into a borehole. The strain gauges detect compressive and tensile displacements along the bar, which are processed as strains to determine the location of shear planes. The accuracies of calculations of the magnitude and direction of shear are dependent on the orientation of the shear strip relative to the direction of shear.

#### 12.7.7.4 Tilts and Slopes

Changes in tilt, or slope, of a surface can be determined from precise surveying and from instrumentation known as tiltmeters. Inclinometers are used to monitor gradient changes within the rock mass. These basically comprise a casing fitted with internal tracks that is permanently grouted into the rock mass. A sensor is periodically run along the tracks to monitor changes in the inclination of the casing as a means of detecting horizontal displacement and shear zones. The instrument is one-dimensional and regard has to be had to the likely direction of movement when orientating the casing during installation.

#### 12.7.7.5 Groundwater Pressure

A piezometer is a device for measuring water pressure. In its simplest form it comprises an open-end standpipe fitted with a strainer that is sealed into a borehole at the target horizon. The height to which water rises in the standpipe is a measure of water pressure at the target horizon. More sophisticated piezometers are based on a sensitive pressure transducer being sealed in at the target horizon. This approach enables a string of transducers to be used to monitor multiple horizons within the one borehole.

### 12.7.8 Field Monitoring Practices

Given the wide range of purposes for which monitoring may be undertaken and variability in site-specific conditions, there are no fixed designs or layouts for undertaking monitoring. Nevertheless, installation practices and monitoring schedules tend to be similar. The following two summary case studies are presented to illustrate some of the concepts associated with designing monitoring layouts.

The first case study relates to a very comprehensive array of monitoring instrumentation installed at the face of a longwall gateroad and monitored over a period of 277 days until the installation was lost due to caving associated with the passage of the longwall face (Lu 2001). The major objective of the investigation was to develop a detailed understanding of, firstly, the deformation behaviour of laminated, weak roof strata under both first workings and abutment loading conditions; and, secondly, the performance of the rock bolting reinforcement system in use at the study site.

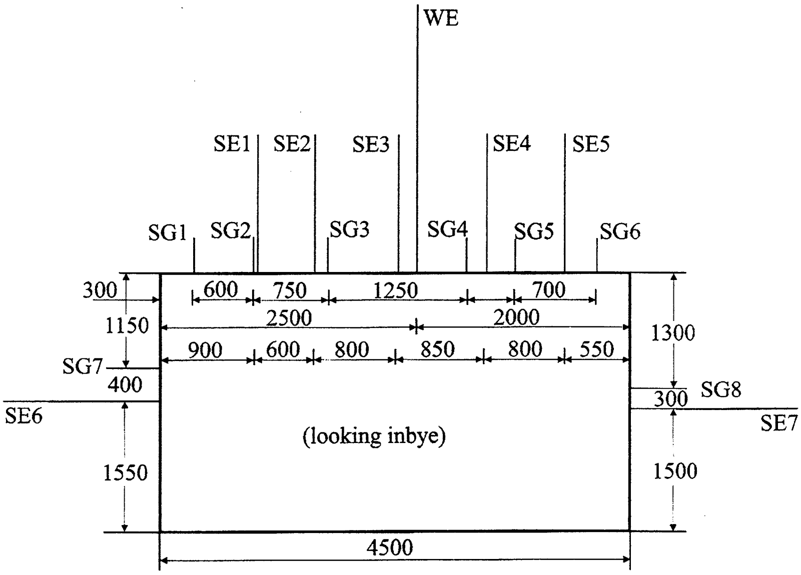
The type and configuration of the monitoring instrumentation utilised in the study is shown in Fig. 12.18. It was installed within 2 m of the mining face in order to measure the maximum components of deformation. The monitoring system was designed such that a profile of roof deformation could be obtained across the full width of the roadway.

The instrumentation comprised:

- A 15 m long, 4 anchor, wire extensometer positioned in the centre of the roadway. This was installed because previous experience at the mine (Angus Place Colliery, Australia) had indicated that deformation could extend beyond the 7.5 m upper limit of a sonic probe. The reference anchor was located at the 15 m horizon and the other anchors at the 12 m, 9.5 m and 7.5 m horizons. The depth of the shallowest anchor was chosen to coincide with the depth of the deepest sonic probe anchor in an immediately adjacent hole.
- 5 sonic probe roof extensometers distributed across the full roadway width, each with 20 anchors that were spread up to 500 mm apart over a length of 7.5 m.
- 6 strain gauged bolts distributed across the full roadway width, each 2.1 m long and fitted with 9 pairs of strain gauges at 200 mm intervals.
- 2 strained gauged bolts, one in each rib.
- 2 sonic probe extensometers, one in each rib.

Based on this monitoring scheme, it was determined that measureable mining-induced deformation extended to a height of at least 12 m, or almost 2.5 times the roadway width, whilst the major deformation zone extended to a height of

**Fig. 12.18** Detailed instrumentation layout installed at the face of a longwall gateroad development: *WE* wire extensometer, *SE* sonic extensometer, *SG* strain gauged bolt (After Lu 2001)



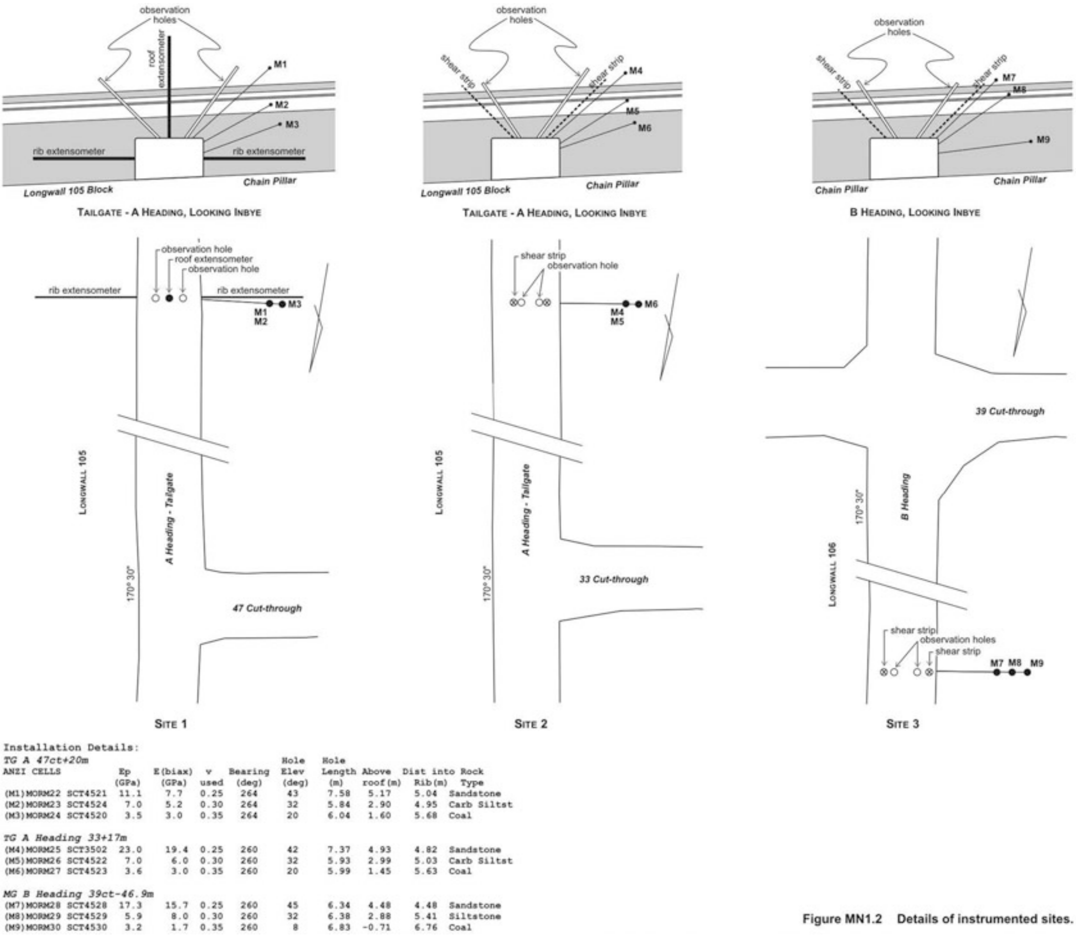
around 6 m. Furthermore, the majority of the differential movement (+65 mm), or dilation, occurred within the immediate coal/claystone roof (0–2 m) and in a mudstone band at the 4–4.5 m horizon. The results indicated that some benefit could be derived from increasing primary bolting length from 2.1 to 2.5 m and that in circumstances where deformation of the upper strata needed to be reduced, long tendons (>6 m) should be installed at the coal face rather than later as secondary support. The monitoring results also suggested that there might be benefit in employing a non-symmetrical reinforcement pattern.

Figure 12.19 relates to the second case study, which consisted of three instrumentation layouts designed by Fabjancyk et al. (2006) to investigate stress changes in stone and coal roof about a gateroad; to measure pillar loads; and to quantify shear stresses developed on bedding planes in the immediate roof as the sites were mined past by a longwall. The magnitude and direction of mining-induced stresses at seam level and in two different strata in the immediate roof were monitored utilising ANZI stress measurement cells. A combination of shear strips and borescope holes was used to identify the location of shear planes and the progress of shearing. Extensometers were installed into the ribs at one site to monitor the development of pillar dilation. A roof extensometer was used at the same site to monitor the development of delamination and softening of the immediate roof above the centre of the roadway.

When instrumentation that is not read with a continuous data logger is installed at a working face for the purpose of monitoring the development of deformation within the immediate roof, a typical reading frequency is:

- immediately after installation at the face;
- at least 2–3 times during the first 5 m of face advance;
- every 10 m of face advance until readings stabilise, but more frequently if movement is accelerating; and
- weekly once movement has stabilised.

Similarly, as a secondary extraction face approaches instrumentation that is not read with a continuous data logger, a typical reading schedule is face distances of 200 m, 100 m, 50 m, 25 m, 10 m, 5 m, 3 m, 2 m, 1 m and 0 m from the approaching face (provided that it is safe to be in the area in cases where the instruments cannot be read remotely).


observation holes
roof extensometer
M1
M2
M3
rib extensometer
rib extensometer
Longwall 105 Block
Chain Pillar
TAILGATE - A HEADING, LOOKING INBYE
shear strip
shear strip
M4
M5
M6
Longwall 105 Block
Chain Pillar
TAILGATE - A HEADING, LOOKING INBYE
M7
M8
M9
Chain Pillar
Chain Pillar
B HEADING, LOOKING INBYE
observation hole
roof extensometer
observation hole
rib extensometer
rib extensometer
LONGWALL 105
A Heading - Tailgate
170° 30'
47 Cut-through
SITE 1
observation hole
33 Cut-through
SITE 2
39 Cut-through
B Heading
LONGWALL 106
observation holes
SITE 3

Installation Details:

*TG A 47ct+20m*

| ANZI CELLS | Ep (GPa) | E(biax) (GPa) | v used | Bearing (deg) | Hole Elev (deg) | Hole Length (m) | Above roof(m) | Dist into Rib(m) | Rock Type |
|---|---|---|---|---|---|---|---|---|---|
| (M1)MORM22 SCT4521 | 11.1 | 7.7 | 0.25 | 264 | 43 | 7.58 | 5.17 | 5.04 | Sandstone |
| (M2)MORM23 SCT4524 | 7.0 | 5.2 | 0.30 | 264 | 32 | 5.84 | 2.90 | 4.95 | Carb Siltst |
| (M3)MORM24 SCT4520 | 3.5 | 3.0 | 0.35 | 264 | 20 | 6.04 | 1.60 | 5.68 | Coal |
| *TG A Heading 33+17m* | | | | | | | | | |
| (M4)MORM25 SCT3502 | 23.0 | 19.4 | 0.25 | 260 | 42 | 7.37 | 4.93 | 4.82 | Sandstone |
| (M5)MORM26 SCT4522 | 7.0 | 6.0 | 0.30 | 260 | 32 | 5.93 | 2.99 | 5.03 | Carb Siltst |
| (M6)MORM27 SCT4523 | 3.6 | 3.0 | 0.35 | 260 | 20 | 5.99 | 1.45 | 5.63 | Coal |
| *MG B Heading 39ct-46.9m* | | | | | | | | | |
| (M7)MORM28 SCT4528 | 17.3 | 15.7 | 0.25 | 260 | 45 | 6.34 | 4.48 | 4.48 | Sandstone |
| (M8)MORM29 SCT4529 | 5.9 | 8.0 | 0.30 | 260 | 32 | 6.38 | 2.88 | 5.41 | Siltstone |
| (M9)MORM30 SCT4530 | 3.2 | 1.7 | 0.35 | 260 | 8 | 6.83 | -0.71 | 6.76 | Coal |

Figure MN1.2 Details of instrumented sites.

**Fig. 12.19** Details of monitoring instrumentation associated with determining pillar load and shear behaviour around a longwall gateroad (After Fabjancyk et al. 2006)

## 12.8 Concluding Remarks

When dealing with advice relating to ground engineering, it is strongly advisable to:

- ask to see the underpinning data;
- verify the competencies of those with input into formulating the advice;
- be wary of anything that is 'proven'; geotechnical engineering is notoriously imprecise;
- seek independent, third party, peer review;
- where feasible, support risk assessment of complex or interactive mechanisms with numerical modelling and probabilistic analysis.

In many respects, the 1990 generic advice of the Institution of Engineers Australia in its seminal publication *Are You at Risk* (IEAust 1990) is still as relevant to ground engineering as it was then. In particular:

- *Engineers project an image of dealing in 'hard' models, whereas in the main they deal with 'soft' models.*
- *As apparent dealers in 'hard' models, engineers also project the image of providing solutions to problems. Since most engineers*

*actively think of, and talk about, themselves as problem solvers, the image is hardly surprising.*

- *Engineers do not really solve problems. They make choices between options for the deployment of resources in response to a need, in the face of considerable uncertainty and gaps in knowledge. It follows that whatever choice is made, it must be to some extent, wrong.*
- *Engineers are perceived as dealers in '*hard*' models because we have developed an impressive number of scientifically based mathematical models which encode some of our experience to some extent.*
- *We know* (*or should know) that our models are limited in their ability to represent real systems, and we use* (*or should use) them accordingly.*
- *The trouble is that we are so inordinately proud of them that we do not present their limitations to the community, and leave the community with the impression that the models are precise and comprehensive.*
- *Despite the proliferation of the mathematical models of engineering science, the engineering method centres around the use of Heuristics, essentially '*rules of thumb*'.*
- *The technique we apply to our heuristics is design.*
- *A simple example of a heuristic is a factor of safety.*
- *There is little doubt that our use of the term '*factor of safety*' implies not merely '*certainty*' but '*certainty+*'. We know that it means no such thing and we allow the misunderstanding to continue.*
- *Design is the central engineering activity.*
- *Design is a process which combines knowledge with judgement to obtain a desired outcome. Our mathematical models contribute only partially to the process, but we often give the impression that they contribute all.*
- *Judgement is the key to the engineering method. It is the only skill which can appropriately manage a heuristic environment.*

Ground engineering is characterised by pervasive uncertainty. Whilst advances in ground engineering have made enormous contributions to improving costs and productivity in the mining sector and to enabling more extensive and more efficient exploitation of mineral resources, the first and most important consideration in the practice of the discipline must always be the safeguarding of health and safety.

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# Appendix 1: Brief History of Key Developments in Ground Engineering

## Brief History of Ground Engineering

Underground mining dates back to at least 40,000 BC when Neanderthal Man mined haematite in Swaziland (Gregory 1980). The earliest known examples of artificial ground support are pillars constructed from piles of stones in malachite mines worked by the Egyptians in the Middle East from about 1350 BC to 1000 BC (Shaw 2006). Agricola (1556) described how timber posts, caps, lagging and lateral restraints were used in the sixteenth century to provide support for roadways in metalliferous mines.

Although the origins of rock mechanics can be traced back to the work of Coulomb in 1773, subsequent developments in this field were sporadic and mostly confined to laboratory studies. Some of the first investigations of a pseudo-scientific nature into ground control were conducted in Belgium in the 1820s, when a Commission was established to investigate surface cracks and damage to buildings caused by ground subsidence over coal mine workings in the city of Liege. By 1880, a number of empirically based theories to account for vertical displacement above mine workings had been developed in Belgium, Germany, France, Great Britain and the USA. Whittaker and Reddish (1989) provide fuller accounts of the evolution of subsidence engineering.

Hood and Brown (1999), Hoek (2007) and Brown (2011) describe developments in ground engineering in general dating from the 1800s. The authors note that technical reports dating from the nineteenth century were based mainly on qualitative visual observations and it was only towards the end of the nineteenth century that mechanisms of ground pressure and deformation began to be postulated. During the first half of the twentieth century, technical reports that treated rock as an engineering material started to appear. Laboratory studies using both photoelastic and material models of rock were reported, with the production of scientific and engineering information about rock properties and the design and stability of structures in rock accelerating rapidly during the 1930s.

There is mention in the literature of rock bolting having been practiced in the USA before the turn of the twentieth century (Gardner 1971) but it was not until 1943 that literature described the planned systematic use of rock bolts. This was at a lead mine in the USA. By 1949, rock bolts were being used in more than 200 mines in the USA, including coal mines (Bieniawski 1987). Rock bolts were tried for the first time in Australian coal mines in 1949, in the Greta Seam at Elrington No. 2 Colliery in NSW (Gardner 1971). Benefits were immediate (McKensey 1952)

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and rock bolts were progressively introduced into the NSW coal industry.

The coining of the term 'strata control' is attributed to the First International Conference on Rock Pressures held in Liege, Belgium in 1951 (Bieniawski 1987). The first annual US Rock Mechanics Symposium was held in 1956. Nevertheless, by the end of the 1950s, there was still no qualitative rock mechanics design method in general use by the mining industry, with researchers in the intervening years being hampered by a lack of data relating their research results to the physical reality in the field (Salamon 1988).

The laboratory study of the mechanical properties of rock was already reasonably well advanced but, apart from notable exceptions, little was known about the behaviour of rock around mining cavities. In Britain, Professor E.L.J. Potts, who headed a large geomechanics research group at the University of Newcastle upon Tyne, argued at the time that strata control problems will never be understood properly unless meaningful field measurements are undertaken (Salamon 1989). A major campaign of field measurements of strains, displacements and stresses was initiated, necessitating the development of suitable instrumentation (Potts 1957).

The failure of the Malpasset Dam in France in 1959, the collapse of Coalbrook Colliery in South Africa in 1960, and the overtopping of the Vajont Dam in Italy due to a landslide in 1962, resulted in a combined loss of over 3500 lives, with these tragedies leading to major advances in rock mechanics. In 1962, the International Society for Rock Mechanics was established in Salzburg. The PhD thesis of Miklos Salamon, submitted to the University of Durham in the same year, appears to have contained the first proposal for numerical analysis on the basis of mathematical models (Salamon 1962).

Salamon was to report later that he had come to the conclusion in the late 1950s that mathematical modelling is essential in mining because the number of variables is so great that it is entirely impractical to explore experimentally their full range of influences. Moreover, no mathematical model is sufficiently general or complete to incorporate all physical aspects of the rock mass; its geometry, behaviour and support; and the mine layout. Therefore, field experiments are vital in evaluating the efficacy of mathematical models (Salamon 1989).

Brown (2011) restated his view that, by the early 1960s, the subject of rock mechanics, if it wasn't yet fully established, was well on its way to becoming established as an identifiable scientific and engineering discipline. Brown cites the appearance of specialist journals, conferences and societies to support his view, noting that the first issue of the first specialist journal devoted to rock mechanics and rock engineering was published in Vienna in 1929.

One of the early handbooks relating to the application of rock mechanics in coal mining was produced in 1976 by Salamon and Oravecz for the South African Chamber of Mines Research Organisation (COMRO) (Salamon and Oravecz 1976). The Foreword to that book reports that up until some 10 years earlier, rock mechanics and strata control in underground coal mines were based largely on rule of thumb methods and some guess work.

During the period from 1960 to 1995, there was a concerted effort to place ground control in underground coal mining on a firm scientific and engineering footing through the establishment of a number of large research institutions in the major coal producing countries of the world. In the Preface to the 1986 edition of *Coal Mine Ground Control,* Peng expressed the view that there was a gap between theory and practice that needed bridging in order to advance the 'art' of ground control into the 'science' of ground control and that this gap was the most urgent and challenging task facing rock mechanics engineers (Peng 1986). As of 1997, Hudson and Harrison were still of the opinion *that although rock mechanics and the associated principles are a science, their application is an art* (Hudson and Harrison 1997).

Hood and Brown (1999) considered that the renaissance period, in the sense of a period of vigorous artistic and intellectual activity, for the field of mining rock mechanics, and perhaps rock mechanics generally, was from about the beginning of the 1960s to about 1983. While the knowledge base created during this renaissance period has been extended, the main emphasis in the post-1983 era has been in the application of the knowledge.

The 1990s and early 2000s were characterised in the western world by the closure of most of the renowned mining research establishments and the demise of a number of minerals tertiary education institutes (Wagner 1999; Wagner and Fettweis 2001; Galvin and Carter 2003). Many of these, such as the South African Chamber of Mines Research Organisation and the National Coal Board in the UK, had a strong basic and applied research focus on fundamental ground engineering principles. During the same period, there was a significant growth in ground engineering research in China.

Table A1.1 summarises the more important developments, milestones and points of note related to underground mining and, in particular, ground engineering since 1770. In the last 50 years, advances in rock mechanics largely account for advances in the theoretical knowledge base that underpins ground control. The more important of these advances have been:

- Recognition that the mode of in situ rock failure is controlled by both the properties of the rock itself and by the load-deformation characteristics of the surrounding rock mass, or loading system. This, in turn, has led to a mechanistic understanding of controlled and uncontrolled rock failure and recognition that rock still retains a substantial load carrying capacity after being loaded beyond its point of maximum resistance to deformation.
- Recognition that the load-deformation behaviour of a rock mass beyond the fractured skin of an excavation can be simulated approximately by a linear elastic model.
- Advances in computational power and developments in numerical modelling software codes, thereby enabling increasingly complex mining situations to be simulated.
- Developments in understanding the mechanics of blocky rock masses, in methods of analysis for blocky jointed rock, and in applying outcomes to excavation engineering and ground support and reinforcement design.

These theoretical advances have been complemented with advances in field instrumentation and monitoring, support technologies and mining techniques, with notable applications in underground coal mining being:

- techniques for measuring in situ stress;
- microseismics to detect failure deep within the rock mass;
- microprocessor monitoring of instrumentation and mining equipment to provide continuous and real time information as to ground response;
- technologies for internally reinforcing the rock mass to improve its self-supporting capacity; and
- static analysis, kinematic configuration, and control and monitoring of longwall powered supports.

**Table A1.1** Timeline of some of the more important developments, milestones and points of note from 1770 related to underground mining and, in particular, ground engineering

| | |
|---|---|
| 1770 | Longwall mining introduced in Britain (hand mining on the advance) |
| 1842 | Employment of women and children in underground mines banned in Europe |
| 1850 | Compressed air first used underground |
| 1851 | Great Britain was the world's largest coal producer, producing about 60 million tonnes per annum |
| | Following the Hartley Colliery disaster of 1862, all coal mines in Great Britain were required to have two separate shafts or other means of access and egress |
| | The annual death rate from falls of ground in Great Britain underground coal mines was 2.02 per 1000 persons employed underground (Siddall 1915) |
| | Annual death rate from all causes was 4.29 per 1000 (Atkinson 1895) |
| 1860s | Pneumatic drill developed and applied to mining |
| | Dynamite invented (1863) |
| | Backfill used to control surface subsidence (1864) |
| | Development of the first mechanical coal cutter, introduced in Great Britain (1868) |
| 1894 | First mechanisation introduced into the Newcastle Coalfield, Australia, at Hetton Colliery in the form of a Jeffrey pneumatically powered coal under-cutting machine |
| 1895 | At a depth of 970 m, the 180 Gold Mine in Bendigo, Victoria, Australia, was reported to be the deepest mine in the world |
| | Falls of ground accounted for 40.5 % of all deaths in Great Britain underground coal industry, nearly double that of the next highest cause, namely explosions (Atkinson 1895). This equated to some 400 deaths from falls of ground |
| 1908 | The annual death rate from falls of ground in underground coal mines in Great Britain was 0.74 per 1000 persons employed underground (Siddall 1915) |
| | A committee appointed to the '*Inquiry into the Causes of and Means of Preventing Accidents from Falls of Ground, Underground Haulages, and in Shafts*' found that the longwall system was the best method of working coal seams (Siddall 1915) |
| 1913 | Falls of ground accounted for 620 deaths and 62,094 accidents in underground coal mines in Great Britain, corresponding to an annual death rate of 0.68 per 1000 persons employed underground (Siddall 1915) |
| 1943 | Apparent first mention in literature of the systematic use of rock bolts, being at a lead mine in the USA |
| 1949 | Rock bolts in use at over 200 mines in the USA |
| | Rock bolts trialled for the first time in an Australian coal mine, at Elrington Colliery, NSW |
| Early 1950s | First continuous miner in Australia, a Joy 1CM, was installed at Newstan Colliery, NSW |
| | Rock bolting progressively introduced into Australian coal and metalliferous mines |
| | Nearly all longwall operations in Great Britain and Europe fitted with hydraulic powered supports |
| Mid 1950s | Shotcrete introduced into the underground construction industry in the mid 1950s (in Europe) |
| | Borehole extensometers developed and used extensively |
| Early to mid 1960s | A range of closed-form solutions existed for stress induced around underground excavations of simple shapes |
| | Problems involving complex shapes were studied using photo-elastic models |
| | Site characterisation (rock mass classification) started to be used in some underground mines |
| | First cable bolts introduced in underground mining in metalliferous operations. This was in South Africa and Canada |
| | First shortwall mining system, comprising hydraulic supports and a continuous miner was installed in Australia at Burwood Colliery |
| | First trials of longwall mining system comprising self advancing powered supports, AFC and a coal plough undertaken in Australia at Coalcliff Colliery |
| | Techniques developed to monitor seismic response of rock mass to mining activity |
| Late 1960s | Electrical resistance analogue computer developed |
| Early 1970s | Cable bolts introduced in underground metalliferous mining in Australia |
| | Servo-controlled rock testing machines developed |
| Mid 1970s | Rock mass classification systems used extensively in the metalliferous mining sector |
| | Microseismic monitoring implemented to determine the location and magnitude of failures within the rock mass |

(continued)

**Table A1.1** (continued)

| | |
|---|---|
| 1984 | Mobile breaker line supports (mobile roof supports) introduced in pillar extraction |
| 1990s | Rapid uptake in use of remote controlled mining equipment in all forms of underground mining |
| | Cable bolts utilised as primary support in underground coal mining |
| | Closure of world-renowned ground control research organisations and tertiary minerals education institutions in the western world and an expansion of ground control research in China |
| 2000–present | Continuing step advances in computational techniques |
| | Continuing advances in ground support and reinforcement systems |
| | Periods of up to several years without a fatal fall of ground in Australian underground coal mines |

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# Appendix 2: Equivalent Moduli for a Stratified Rock Mass

## Equivalent Elastic Moduli

Solution for the equivalent moduli of stratified rock mass (Salamon 1968).

If $E_{eq-n}\, and\, E_{eq-p}$ are equivalent Young's moduli in the plane of transverse isotropy and in the direction normal to it, respectively, then:

$$\frac{1}{E_{eq-n}} = \frac{\sum \frac{\Phi_j E_j}{1 - \nu_j^2}}{\sum \frac{\Phi_j E_j}{1 + \nu_j} \sum \frac{\Phi_j E_j}{1 - \nu_j}} \tag{A2.1}$$

$$\frac{1}{E_{eq-p}} = \sum \Phi_j \left( \frac{1}{E_j'} - 2\left[\nu_j'\right]^2 . \frac{E_j}{E_j'} . \frac{1}{1 - \nu_j} \right) + 2\left( \frac{\left[\sum \Phi_j . \frac{E_j}{E_j'} . \frac{\nu_j'}{1-\nu_j}\right]^2}{\frac{\sum \Phi_j E_j}{1 - \nu_j}} \right) \tag{A2.2}$$

where

*$\nu$ and $\nu'$ are Poisson's ratios in the plane of transverse isotropy to a stress acting parallel and normal to it, respectively.*

$E_j, \nu_j, h_j =$ *Elastic modulus, Poisson's ratio, and thickness of $j^{th}$ layer, respectively*

$T = total\, strata\, thickness$

$$\Phi_j = \frac{h_j}{T}$$

## Reference

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# Appendix 3: Basic Statics Formulations for a Clamped and a Simply Supported Beam Subjected to Transverse Load

## Terminology

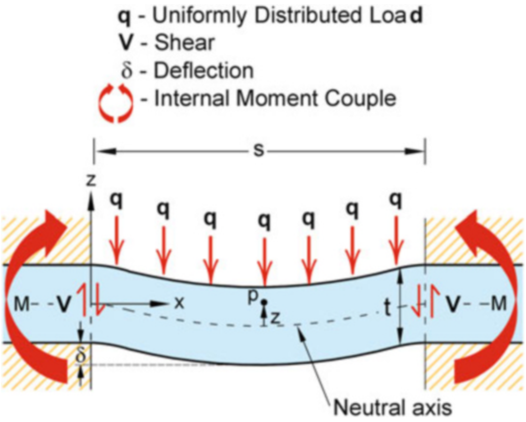

**Table A3.1** Tabulation of symbols

| Symbol | Parameter |
|---|---|
| p | point of interest |
| s | span of beam (m) |
| x | longitudinal distance from end of beam to point p (m) |
| t | thickness of beam (m) |
| z | normal (transverse) distance from neutral axis to point p (m) |
| q | load per unit width acting on beam (N) |
| $\gamma$ | unit weight of beam (N/m$^3$) |
| $\delta_x$ | deflection of beam at a distance of ‘x’ from the beam end (m) |
| E | elastic modulus of beam (N/m$^2$) |
| I | moment of inertia (m$^4$) |
| $M_x$ | bending moment at a distance of ‘x’ from the beam end (N m) |
| $\sigma_x$ | bending stress at a distance of ‘x’ from the beam end (N/m$^2$) |
| $V_x$ | shear force at a distance of ‘x’ from the beam end (N) |
| $\tau_{xz}$ | shear stress generated by shear force $V_x$ (N/m$^2$) |

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## Clamped Beam

**Table A3.2** Formulations and maximum values for deformation parameters associated with a uniformly loaded, clamped beam of rectangular cross-section and unit width

| Parameter | Formula | | Maximum value | |
|---|---|---|---|---|
| Deflection $\delta_x$ | $\delta_x = \frac{qx^2(s-x)^2}{24EI}$<br>$= \frac{\gamma x^2(s-x)^2}{2Et^2}$ | Eq. A3.1 | $\frac{\gamma s^4}{32Et^2}$ at $\frac{s}{2}$ | Eq. A3.2 |
| Bending Moment $M_x$ | $M_x = \frac{q(6sx - 6x^2 - s^2)}{12}$ | Eq. A3.3 | $-\frac{qs^2}{12}$ at abutment | Eq. A3.4 |
| Moment of Inertia | $I = \frac{bt^3}{12}$<br>$= \frac{t^3}{12}$ for unit width | Eq. A3.5 | – | – |
| Bending Stress $\sigma_x$ | $\sigma_x = \frac{M_x z}{I} = \frac{12M_x z}{t^3}$ | Eq. A3.6 | $\frac{qs^2}{2t^2} = \frac{\gamma s^2}{2t}$ at abutment | Eq. A3.7 |
| Shear Force $V_x$ | $V_x = q\left(\frac{s}{2} - x\right)$ | Eq. A3.8 | $\frac{qs}{2}$ at abutment | Eq. A3.9 |
| Shear Stress $\tau_{xz}$ | $\tau_{xy} = \frac{3V_x}{2}\left(\frac{t^2 - 4z^2}{t^3}\right)$ | Eq. A3.10 | $\frac{3qs}{4t} = \frac{3\gamma s}{4}$ in neutral axis at abutments | Eq. A3.11 |

## Simply Supported Beam

**Table A3.3** Formulations and maximum values for deformation parameters associated with a uniformly loaded, simply supported beam of rectangular cross-section and unit width

| Parameter | Formula | | Maximum value | |
|---|---|---|---|---|
| Deflection $\delta_x$ | $\delta_x = \frac{qx(s^3 - 2sx^2 + s^3)}{24EI}$<br>$= \frac{\gamma x(s^3 - 2sx^2 + s^3)}{2Et^2}$ | Eq. A3.12 | $\frac{5\gamma s^4}{32Et^2}$ at $\frac{s}{2}$ | Eq. A3.13 |
| Bending Moment $M_x$ | $M_x = \frac{qx(s-x)}{2}$ | Eq. A3.14 | $-\frac{qs^2}{8}$ at $\frac{s}{2}$ | Eq. A3.15 |
| Moment of Inertia | $I = \frac{bt^3}{12}$<br>$= \frac{t^3}{12}$ for unit width | – | – | – |
| Bending Stress $\sigma_x$ | $\sigma_x = \frac{M_x z}{I} = \frac{12M_x z}{t^3}$ | Eq. A3.16 | $\frac{3qs^2}{4t^2} = \frac{3\gamma s^2}{4t}$ at $\frac{s}{2}$ | Eq. A3.17 |
| Shear Force $V_x$ | $V_x = q\left(\frac{s}{2} - x\right)$ | Eq. A3.18 | $\frac{qs}{2}$ at abutment | Eq. A3.19 |
| Shear Stress $\tau_{xz}$ | $\tau_{xy} = \frac{3V_x}{2}\left(\frac{t^2 - 4z^2}{t^3}\right)$ | Eq. A3.10 | $\frac{3qs}{4t} = \frac{3\gamma s}{4}$<br>in neutral axis at abutments | Eq. A3.11 |

## Beam Loaded by a Less Stiff Beam

The analysis of the interaction between two beams of rectangular cross-section where the lower beam is stiffer and therefore loaded by the upper beam, is based on the following assumptions:

1. Each beam is homogenous, isotropic and elastic.
2. The two beams are of equal width, b, and length, l.
3. The cohesion and friction between the two beams is zero.
4. The deflection of the two beams is equal over the full length of the beams.

5. The upper beam loads the lower beam with a uniform load per unit length.
6. The lower beam supports the upper beam with an equal load per unit length.

The deflection of the two clamped beams at any point, x, along the length of the beams is given by:

$$\delta_l = \delta_u = \frac{(q_l + \Delta q)}{24E_l I_l}\left(x^2\right)(l-x)^2 = \frac{(q_u - \Delta q)}{24E_u I_u}\left(x^2\right)(l-x)^2 \quad \text{(A3.12)}$$

where

*subscripts 'l' and 'u' denote lower and upper beam, respectively*

$\Delta q$ = *load transfer to lower beam*

Since Eq. A3.12 holds true for all values of x, it follows that

$$\frac{(q_l + \Delta q)}{24E_l I_l} = \frac{(q_u - \Delta q)}{24E_u I_u} \quad \text{(A3.13)}$$

and so

$$\Delta q = \frac{q_u E_l I_l - q_l E_u I_u}{E_l I_l + E_u I_u} \quad \text{(A3.14)}$$

Therefore, new loading on lower beam is

$$q_l' = q_l + \Delta q = \frac{E_l I_l (q_l + q_u)}{E_l I_l + E_u I_u} = \frac{E_l t_l^3 (q_l + q_u)}{E_l t_l^3 + E_u t_u^3} \quad \text{(A3.15)}$$

Substituting Eq. A3.12 into Eq. A2.1 gives:

$$\delta_l = \delta_u = \frac{(q_l + q_u)x^2}{24(E_l I_l + E_u I_u)}(l-x)^2 \quad \text{(A3.16)}$$

Since

$$\begin{aligned} q_l &= \rho_l g b t_l \\ q_u &= \rho_u g b t_u \\ I_l &= \frac{b t_l^3}{12} \\ I_u &= \frac{b t_u^3}{12} \end{aligned}$$

Then

$$\delta_l = \delta_u = \frac{g(\rho_l t_l + \rho_u t_u)}{2\left(E_l t_l^3 + E_u t_u^3\right)} x^2 (l-x)^2 \quad \text{(A3.17)}$$

Equation A3.17 can be used to estimate the average load on the lower stratum. It is likely to be conservative because it takes no account of resistance to bending provided by friction between layers and, therefore, numerical modelling is advisable in high consequence situations.

# Appendix 4: Foundation Behaviour

## Introduction

In this text, a coal pillar is considered to constitute a **footing**, with the strata immediately above and below the pillar constituting **foundations**. All strata comprising foundations are referred to generically as **layers**. The term **soft** refers to materials that are more soil-like and homogenous with little in the way of defects, so that the low uniaxial compressive strength (UCS) of these materials is due to the low strength of the intact material. The term **weak** refers to materials that are not necessarily homogenous and have a low strength, typically in a UCS range of 0.5 to 10 MPa, as a result of the very low strength of the intact material and/or because of a significant density of lower strength defects.

Civil engineering foundation behaviour principles are discussed under the headings of

- Settlement
- Ultimate Bearing Capacity
- Creep
- Rebound (swell)

The terms **creep** and **swell** have different meanings in civil engineering foundation theory to their colloquial uses in underground coal mining.

## Fundamentals

### Undrained and Drained Behaviour

The presence of water significantly influences the behaviour of foundation materials, especially those that are soil-like or of low strength. Settlement and ultimate bearing capacity are a function of this behaviour.

The strength of a dry soil mass is due to:

- adhesion between particles; and
- friction between particles.

Soil behaviour can be described by the Mohr failure criterion. Sand-based soils are characterised by relatively low cohesion and relatively high friction whilst the reverse is generally true for clay-based soils. Figure A4.1 illustrates the effect of saturating a soil-like material. It is assumed that the presence of water does not affect the cohesive strength of the soil. When the material is loaded, the increase in normal stress, $\sigma_n$, across the particles is opposed by an increase in hydrostatic pressure, u, which acts to keep the particles apart. The actual or effective normal stress, $\sigma'_n$, in a saturated material is given by Eq. A4.1.

$$\sigma_n^{'} = \sigma_n - u \qquad \text{(A4.1)}$$

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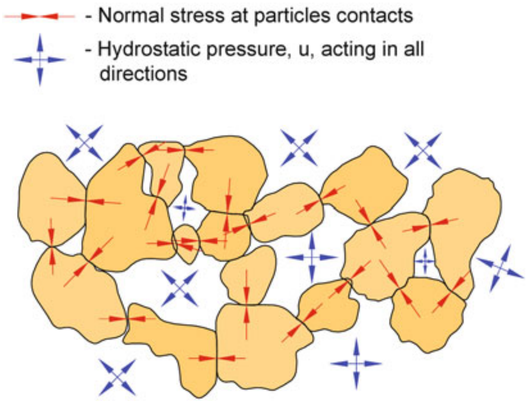
- Normal stress at particles contacts
- Hydrostatic pressure, u, acting in all directions

**Fig. A4.1** Normal and hydrostatic stress components in a saturated soil-like material

An increase in load acts to force the particles closer together. This is resisted if pore water cannot be immediately dissipated, resulting in some or all of the additional load being carried by the incompressible pore water. The time taken for this pressure to dissipate is determined by its coefficient of consolidation, $C_v$, and the distance that the pore water has to travel through the material before it reaches a free draining surface. Coefficient of consolidation is defined by Eq. A4.2, which shows that it is a function of the coefficient of permeability, or hydraulic conductivity, K, of the material and the compressibility of the material as described by the coefficient of volume compressibility, $m_v$. The material is said to be in an **undrained** state until such time as pore water pressure is restored to a steady state, when it is then referred to as being in a **drained** state.

$$C_v = \frac{K}{m_v \gamma_w} \quad \left(m^2/s\right) \tag{A4.2}$$

where

$K = hydraulic\ conductivity\ (m/s)$
$m_v = coefficient\ of\ volume\ compressibility\ (m^2/N)$
$\gamma_w = unit\ weight\ of\ water\ N/m^3$

In the case of a clay material, permeability is low for a given value of coefficient of volume compressibility and so the coefficient of consolidation is also low. Therefore, this material is slow draining. However, whilst a rock-like material may have the same low permeability as a clay, its compressibility is likely to be two or three orders of magnitude lower. This means that the coefficient of consolidation for rock-like material is two to three orders of magnitude higher than that of a clay material of the same permeability and, hence, that pore pressures will dissipate 100 to 1000 times quicker than in the clay material. This is a critical consideration in coal mining environments because foundation materials can cover the full spectrum from clay through to massive, strong rock. Therefore, lithologies and material properties need to be well understood when applying civil engineering bearing capacity approaches to the design and assessment of coal pillar foundations.

The overall effect of this behaviour is that fast draining materials are considered to behave in a drained manner when loaded, with the result that the predominant frictional strength increases with increased loading. On the other hand, slow draining materials such as clay are considered to behave in an undrained manner when loaded and, since their strength is attributable primarily to cohesion, they exhibit little change in strength with increased loading.

## Consolidation

In the process of draining, particles move closer together as pore water pressure is dissipated, with consolidation resulting in a progressive increase in shear strength due to increase in effective stress and increased friction between particles. Consolidation settlement is initially proportional to the square root of time, before transitioning to being exponentially-based in later stages. Therefore, it is common practice to base consolidation analysis on the time taken to reach 90 % or 95 % consolidation. The relationship between time and consolidation for situations where pore pressure is initially uniform in a consolidating layer is given by Eq. A4.3.

$$T_x = T_{F(x)} \frac{H^2}{C_v} \tag{A4.3}$$

where

$T_x$ = time to reach x% consolidation (seconds)
$T_{F(x)}$ = time factor associated with x% consolidation
= 0.196 for 50% consolidation
= 0.818 for 90% consolidation
= 1.125 for 95% consolidation
$C_v$ = the coefficient of consolidation ($m^2/s$)
$H$ = the length of the drainage path ($m$)

Soil-like materials that have been unloaded to some degree with the passage of geological time are referred to as being **over-consolidated**. Unloading results in an increase in the volume of these materials, referred to as either **rebound** or **negative consolidation**.

In coal mining environments, the undrained and drained moduli of soil-like foundation materials such as underclay and degraded claystone can be one to two orders of magnitude lower than those of the surrounding strata. Hence, settlement of this material can result in significant roof to floor convergence, leading to elevated levels of surface subsidence which have sometimes been misinterpreted as indicating an unstable pillar system.

## Settlement

Three components contribute to the settlement of soil-like strata above and below a coal pillar, namely:

- **immediate settlement**, or undrained settlement, due to deformation of the microstructure of the foundation material without a change in its volume;
- **primary consolidation settlement**, resulting from dissipation of pore water pressure as the foundation material moves from an undrained to a drained state; and
- **secondary consolidation settlement**, due to deformation under the effects of constant effective stress and often referred to as **creep.**

Immediate settlement is also referred to as **elastic settlement**, although it can include an inelastic component. The elastic component comprises the displacement, or strata compression, that occurs in response to the increased stress induced in the roof and floor strata when a pillar is formed. The inelastic component of immediate settlement arises if, during the process of draining, the soil structure goes into yield (but not bearing capacity failure). The calculation of stress and strain distributions during and after yield requires the use of numerical models, such as finite element analysis. On occasions, the inelastic component can be significantly higher than the elastic component (Vasundhara, 1999).

A general formula for elastic settlement, $S_e$, is given by Eq. A4.4 which, when applied to a coal pillar, translates into Eq. A4.5. The influence factor, $I_p$, depends on Poisson’s ratio, the ratio of the thickness of the foundation layer, t, to the width of the footing, w, and the shape of the footing. A number of theoretical solutions have been developed for elastic settlement, giving rise to a range of influence factors.

$$S_e = \frac{qBI_p}{E_u} \tag{A4.4}$$

where

$S_e$ = elastic settlement ($m$)
$B$ = footing diameter or width ($m$)
$q$ = footing working stress ($Pa$)
$E_u$ = undrained modulus ($Pa$)
$I_p$ = a dimensionless influence factor, which is a function of footing flexibility, footing width, footing length, thickness of foundation layer and Poisson’s ratio

$$S_e = \frac{\sigma_{ips} w_{\min} I_p}{E_u} \tag{A4.5}$$

where

$\sigma_{ips}$ = induced average pillar stress ($Pa$)
= average pillar stress minus vertical component of primitive stress
$w_{min}$ = minimum pillar width ($m$)

Long term settlement, or primary consolidation, is a function of the compressibility of the material being loaded and its existing state of compaction, both of which are dependent on the stress history of the material. Equation A4.6 defines the general solution for settlement of normally consolidated material as given by Das (1998).

$$S_p = \frac{C_c t}{1+e_o} log \frac{\sigma_{ve} + \Delta\sigma_{vi}{}^{vivi}}{\sigma_{ve}} \qquad (A4.6)$$

where

$S_p = primary\ consolidation\ settlement\ (m)$
$C_c = compressibility\ index$
$t = foundation\ layer\ thickness\ (m)$
$e_o = void\ ratio\ (a\ measure\ of\ the\ existing\ state\ of\ compaction)$
$\sigma_{ve} = vertical\ effective\ stress\ (N/m^2)$
$\Delta\sigma_{vi} = induced\ vertical\ stress\ (N/m^2)$

Many highly over-consolidated materials behave approximately as linear elastic materials within the working stress ranges encountered in civil engineering practice. Therefore, Eq. A4.4 is often extended to these situations by substituting a drained value for modulus and selecting influence functions based on the value of Poisson's ratio for the drained state. Some procedures incorporate additional and/or alternative influence functions.

**Secondary consolidation** involves the ongoing readjustment of material particles under sustained load. The amount of **secondary settlement**, $S_s$, that occurs over time is directly proportional to the thickness of the foundation layer after primary consolidation and its **secondary compression index**, $C_\alpha$, with this relationship given by Eq. A4.7.

$$S_s = C_\alpha t_{100} \log\left(\frac{T_1}{T_2}\right) \qquad (A4.7)$$

where

$S_s = secondary\ consolidation\ settlement\ (m)$
$C_\alpha = secondary\ compression\ index$
$t_{100} = foundation\ layer\ thickness\ at\ end\ of\ primary\ consolidation\ (m)$
$T_1 = time\ when\ secondary\ consolidation\ is\ assumed\ to\ begin\ (s)$
$T_2 = time\ for\ which\ secondary\ settlement\ is\ calculated\ (s)$

## Bearing Capacity

Bearing capacity theory is premised on the assumption that upon exceeding a certain stress condition, rupture surfaces will develop in the foundation material and it will fail in shear. In situations where the foundation material is homogenous over a thickness of two to three times the width of the footing, failure is generally considered to take one of three progressive forms:

- Punch shear failure
- Local shear failure
- General shear failure

Punch shear failure occurs when compression of the foundation material is sufficient to cause shear failure surfaces to develop beneath the footing. These delineate a wedge of material, corresponding to Zone 1 in Fig. A4.2, that is then driven into the ground, resulting in compaction. There may be little movement of the ground on the sides of the footing and no visible signs of failure. This behaviour is more likely to occur in very compressible materials such as loose sands and weak clays.

Local shear failure is an extension of punch shear failure in which compression of the

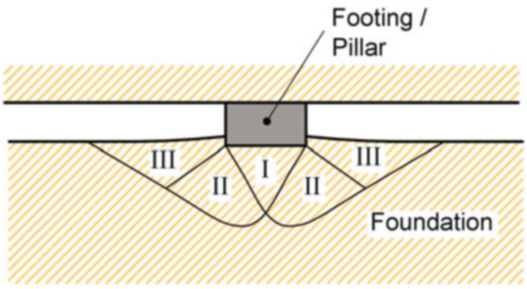

**Fig. A4.2** Bearing capacity shear rupture zones

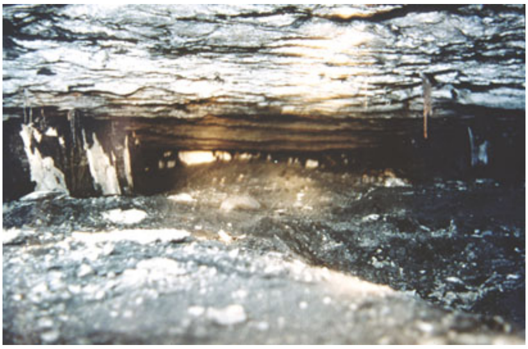

**Fig. A4.3** Bearing capacity failure beneath abutment pillars in partial pillar extraction workings, resulting in some 2 m of closure in 2.7 m high workings

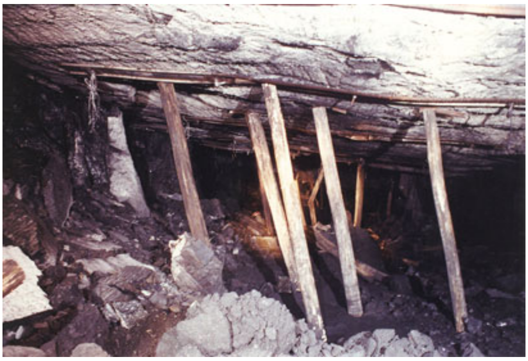

**Fig. A4.4** Extrusion of foundation material from beneath a pillar as evidenced by the angle of the props and the condition of the pillar side

foundation material leads to the formation of slip failure surfaces outside of the perimeter of the footing. These correspond to Zone 2 in Fig. A4.2. The shear failure surfaces do not daylight, although the surface around the footing may bulge slightly.

General shear failure is the most common type of shear failure and is characterised by well-defined slip surfaces that extend from one edge of the footing and daylight on the opposite side of the footing (Fig. A4.2). Materials that are practically incompressible and have finite shear strength fail in general shear. As the material in Zone I moves down, plastic flow is initiated in Zone II. The overlying material in Zone III provides resistance to this displacement but once this is overcome, the shear failure planes extend to the surface and the material around the footing bulges. Figure A4.3 shows an example of general shear failure of the floor adjacent to pillars in a coal mine in the Lake Macquarie region of NSW, Australia.

If the thickness, t, of a weak foundation layer is limited or the layer is significantly weaker than adjacent low strength layers, slip surfaces may not develop throughout the full thickness of the foundation. Rather, the layer can fail, shear on bedding planes and extrude laterally from under (or over) the footing such as in the field case illustrated in Fig. A4.4. When this occurs in coal mining situations, the pillar is subjected to lateral tension and open cracks can develop from roof to floor and extend through the full width of the pillar because the tensile strength of the pillar is minimal, especially in the presence of cleating. The pillar failure mode moves from one of compressive failure under axial load to one of tensile failure under lateral stress. In these circumstances, the distinction between extrusive foundation failure and pillar system failure associated with low cohesion and friction interfaces becomes blurred as both mechanisms are likely to be active and interacting.

In thick foundation situations ($t > 2w$, where w is the width of the footing), it is generally accepted that more rigid and incompressible foundation materials will fail in general shear, whilst more compressible materials will fail in local shear or punching shear. A foundation comprising saturated, normally consolidated clay will fail in general shear if it is loaded so quickly that there is no time for volume change to take place, whilst it may fail in punching shear if the loading rate is sufficiently slow that the material has time to drain and compress (Vesic 1975).

Evaluation of ultimate bearing capacity is usually concerned with general shear failure. Bearing capacity formulae date back to the mid 1850s, with a major advance made when Prandtl (1921) adapted limit equilibrium solutions derived for the penetration of soft, homogenous, isotropic, rigid materials by hard metal punches to assess the bearing capacity of shallow foundations. Buisman (1940) developed this formulation into the Buisman-Terzaghi bearing capacity equation (Terzaghi 1943) for a uniformly loaded, infinitely long strip footing founded on a homogenous incompressible material (Eq. A4.8). Subsequently, this equation has been modified to form the basis of a myriad of bearing capacity formulae.

$$q_u = cN_c + \gamma dN_q + \frac{\gamma w N_\gamma}{2} \tag{A4.8}$$

where

$q_u$ = *ultimate bearing capacity*

$N_c$, $N_q$, $N_\gamma$ = *bearing capacity factors which depend on the value of internal friction*, $\phi$

$w$ = *width of footing*

$\gamma$ = *unit weight of soil*

d = *depth of footing beneath the surface*

Equation A4.8 is made up of three components representing:

- cohesion of the foundation material (‘c’);
- surcharge load acting on the surrounds of the footing (‘γd’); and
- unit weight of the foundation material (‘γ’).

A range of mathematical relationships has been proposed for determining $N_c$, $N_q$, and $N_\gamma$, all of which are a function of ϕ. The variation in values for $N_c$ and $N_q$ is relatively minor, with the values given by Eqs. A4.9 and A4.10 finding extensive application (e.g. Vesic 1975; Smith 1990; Brady and Brown 1993; Craig 1997).

$$N_q = Tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right)e^{\pi \tan\phi} \tag{A4.9}$$

$$N_c = \left(N_q - 1\right)Cot\phi \tag{A4.10}$$

There is around a seven fold range in the values proposed for $N_\gamma$, associated mainly with the manner in which a representative value of ϕ is selected (Vesic 1975). Equations A4.11, A4.12, A4.13, and A4.14 define four expressions which find wider use.

$$N_\gamma = \left(N_q - 1\right)Tan\,(1.4\phi) \qquad Meyerhof\ (1963) \tag{A4.11}$$

$$N_\gamma = 1.5\left(N_q - 1\right)Tan\phi \qquad Brinch - Hansen\,(1970) \tag{A4.12}$$

$$N_\gamma = 2\left(N_q + 1\right)Tan\ \phi \qquad Vesic\ (1975) \tag{A4.13}$$

$$N_\gamma = 2\left(N_q + 1\right)Tan\varphi\ Tan\left(\frac{\pi}{4} + \frac{\phi}{5}\right) \qquad Chen\ \text{and}\ McCarron\,(1990) \tag{A4.14}$$

Equation A4.12 was developed for cohesive, frictional material such as soft rock and overconsolidated clays and has been adopted by Bieniawski (1987) and Brady and Brown (2006) for mine design. Experimental results suggest that it produces lower end values in comparison to other expressions for $N_\gamma$ (Chen and McCarron 1990).

Equation A4.8 assumes the strip footing to be infinitely long and, as such, it is a two-dimensional solution. According to Vesic (1975), a footing can only be assumed to be an infinite strip in a strict sense once its length, L, is greater than 10 times its width, w; practically, however, this can be assumed once L/w > 5. For smaller L/w ratios, semi-empirical shape correction factors have to be introduced and so Eq. A4.8 translates to Eq. A4.15.

$$q_u = cN_cS_c + \gamma dN_qS_q + \frac{\gamma w N_\gamma S_\gamma}{2} \tag{A4.15}$$

where

$S_c$, $S_q$, $S_\gamma$ = *dimensionless shape factors*

The shape factors recommended by Vesic (1975) for rectangular, circular and square footings are presented in Table A4.1. They are

**Table A4.1** Bearing capacity shape factors (after Vesic, 1975)

| Shape | $S_c$ | $S_q$ | $S_\gamma$ |
|---|---|---|---|
| Strip | 1.0 | 1.0 | 1.0 |
| Rectangular | $1 + \left(\frac{L}{w}\right)\left(\frac{N_q}{N_c}\right)$ | $1 + \left(\frac{L}{w}\right)\tan\phi$ | $1 - 0.4\left(\frac{L}{w}\right)$ |
| Circular and square | $1 + \left(\frac{N_q}{N_c}\right)$ | $1 + \tan\ \phi$ | 0.60 |

**Table A4.2** Bearing capacity shape factors (after Terzaghi 1943)

| Shape | $S_c$ | $S_q$ | $S_\gamma$ |
|---|---|---|---|
| Strip | 1 | 1 | 1 |
| Rectangular | $1+0.2\left(\frac{L}{w}\right)$ | 1 | $1-0.2\left(\frac{L}{w}\right)$ |
| Square | 1.2 | 1 | 0.8 |
| Circular | 1.2 | 1 | 0.6 |

not unique, with those recommended earlier by Terzaghi (1943) and shown in Table A4.2 also continuing to find wide acceptance.

Several techniques have been proposed for estimating the ultimate bearing capacity of non-homogenous and anisotropic foundation materials. These include:

- Mandel and Salencon (1969) – a weak deformable layer overlying a layer of infinite rigidity and strength;
- Brown and Meyerhof (1969) and Vesic (1973) – a cohesive two layer model, with undrained conditions in both layers ($\phi = 0$);
- Smith (1990) – a procedure for averaging cohesion and friction values in thin soil layer situations;
- Chugh et al. (1990) – a shallow foundation on a two layered rock system with consideration of cohesion and friction in each layer.

Irrespective of which bearing capacity procedure is invoked, the bearing capacity of a surface foundation (depth of footing, d, = 0) is least when it is fully saturated and in an undrained state, in which case $\phi = 0°$ and $N_\gamma = 0$. In a mining environment in which the roadways have not been backfilled, flooded or affected by roof falls, the surcharge load of any material around the pillar can also be equated to zero; that is, $q = 0$. Hence, Eq. A4.15 reduces to one term, defined by Eq. A4.16.

$$q_u = cN_cS_c \tag{A4.16}$$

The more common formulae which have been applied to mining environments on the basis that $\phi = 0°$ are summarised in Table A4.3.

Some of the assumptions and approximations associated with bearing capacity models that take on added significance in an underground mine setting include:

- Interaction between footings: In civil engineering applications, the spacing between footings is often many times greater than the width of the footings. The converse applies in most underground coal mining environments, giving rise to the potential for pillar foundations to interact. The confinement provided by a pillar to the foundation zone of an adjacent pillar could be expected to retard the development of shear failure, resulting in an increase in bearing capacity. Benefits are difficult to quantify in the field but laboratory studies of strip footings by Stuart (1962) and West and Stuart (1965) revealed that bearing capacity started to increase because of interaction once the distance between footings was less than 3 times the footing width. Bearing capacity peaked at 200 % of that for a single footing when the spacing between the footings was reduced to 0.25 times the footing width.
- Discontinuities: Discontinuities in the foundation material reduce the shear strength of the material mass. Usually, it is not feasible to determine the in situ shear strength of fractured material from laboratory determined values of cohesion and friction. Operational strength can be expected to lie somewhere between laboratory determined shear strength and the shear strength of the fracture planes. Ganow (1975) used a reduction value of 35 % for studies in the Illinois coal basin. The author also applied a reduction factor of 60 % to account for laboratory testing being biased towards stronger, less fractured samples as a result of weak material not surviving the specimen preparation process.
- Groundwater and flooding: Water can affect all three components of bearing capacity. The $cN_c$ component is affected because the shear strength of soft and weak rock and soil materials can be reduced significantly in the presence of water. For example, the unconfined compressive strength of selected, wet, coal-bed floor strata has been found to be only 26 to 40 % of that in its dry state (Bieniawski

**Table A4.3** A selection of bearing capacity formulae which have found application in underground coal mining for the case of $\phi = 0°$

<table>
<tr><th>Theory</th><th>Pillar shape</th><th>Ultimate bearing capacity ($q_u$)</th><th colspan="2">Modification factors</th></tr>
<tr><td>**Classical:**<br>**(Buisman-Terzaghi)**<br>Single homogenous layer</td><td>Strip</td><td>$cN_c = 5.14c$</td><td colspan="2">$N_c = 2 + \pi = 5.14$</td></tr>
<tr><td rowspan="3">**Classical: incorporating shape factors**<br>Single homogenous layer</td><td></td><td>$cN_cS_c$</td><td colspan="2"></td></tr>
<tr><td>Square</td><td>$cN_c\left[1 + \frac{N_q}{N_c}\right] = 6.168c$</td><td colspan="2" rowspan="2">$N_c = 5.14$<br>$\frac{N_q}{N_c} = 0.2$</td></tr>
<tr><td>Rectangular</td><td>$cN_c\left[1 + \left(\frac{L}{w}\right)\left(\frac{N_q}{N_c}\right)\right]$<br>$= c\left[5.14 + 1.028\left(\frac{L}{w}\right)\right]$</td></tr>
<tr><td rowspan="11">**Mandel and Salencon (1969)**<br>Soft layer over a layer of infinite rigidity and strength</td><td rowspan="11">Strip</td><td rowspan="11">$cN_cF_c = 5.14cF_c$</td><td colspan="2">$N_c = 5.14$</td></tr>
<tr><td>**w/t**</td><td>**$F_c$**</td></tr>
<tr><td>≤1.41</td><td>1</td></tr>
<tr><td>2</td><td>1.02</td></tr>
<tr><td>3</td><td>1.11</td></tr>
<tr><td>4</td><td>1.21</td></tr>
<tr><td>5</td><td>1.30</td></tr>
<tr><td>6</td><td>1.4</td></tr>
<tr><td>8</td><td>1.59</td></tr>
<tr><td>10</td><td>1.78</td></tr>
<tr><td colspan="2">Thereafter:<br>$N_cF_c \approx (\pi + 1 + 0.5 w/t)$</td></tr>
<tr><td colspan="5">**Brown and Meyerhof (1969)** $cN_m$ where $N_m$ is a function of $c_2/c_1$</td></tr>
<tr><td rowspan="2">Soft layer overlying a stiffer layer<br>$c_1$, $t_1$ = cohesion and thickness of top layer<br>$c_2$, $t_2$ = cohesion and thickness of bottom layer</td><td>Strip</td><td>$c_1\left[4.14 + 1.1\left(\frac{w}{t_1}\right)\right] for \frac{w}{t_1} > 0.9$</td><td colspan="2"></td></tr>
<tr><td>Square</td><td>$\cong c_1\left[5.05 + 0.66\left(\frac{w}{t_1}\right)\right] for \frac{w}{t_1} > 1.5$</td><td colspan="2">Derived for a circular footing of diameter w.</td></tr>
<tr><td rowspan="2">Stiffer layer overlying a softer layer<br>$c_1$, $t_1$ = cohesion and thickness of top layer<br>$c_2$, $t_2$ = cohesion and thickness of bottom layer</td><td>Strip</td><td>$1.5\left(\frac{t_1}{w}\right)c_1 + 5.14c_2$</td><td colspan="2"></td></tr>
<tr><td>Square</td><td>$\cong 3.0\left(\frac{t_1}{w}\right)c_1 + 6.05c_2$</td><td colspan="2">Derived for a circular footing of diameter w.</td></tr>
</table>

1987). If the foundation material is saturated but the water table is more than a distance of twice the footing width (i.e. >2w) below the footing, the $\gamma qN_q$ component can be based on the unit weight, $\gamma$, of the foundation material. Otherwise, this component should be based on effective material weight, $\gamma'$, in order to account for buoyancy effects. Such effects may be inevitable when mine workings flood and, therefore, need to be taken into account at the design stage. One consequence of mine workings becoming flooded is that a surcharge load is applied to the roof and floor surfaces of the foundations, in which case the $\gamma wN_\gamma$ component may need to be included in bearing capacity calculations.

- Scale effects on strength: Bearing capacity theory does not account for reduced average shear strength along failure slip planes as foundation size increases.

## Creep

In underground coal mining, the term **creep** is used to describe a regional, time dependent failure of the pillar system that results in severe seam convergence, roof and floor failure, rib spall and in some instances, total collapse. In classical engineering, it refers more specifically to the process of continued deformation (strain) of a material under sustained constant load.

Classical engineering creep is a function of deviator stress ($\sigma_1$–$\sigma_3$). Most soft and weak rocks exhibit both immediate (elastic) and delayed (viscous) deformation when subjected to load (Fig. A4.5). Immediate elastic strain is followed by a period of primary creep which develops at a decreasing rate. This may be followed by secondary creep that occurs at a near constant rate. If a material is already approaching its peak strength, secondary creep may develop into tertiary creep, in which case strain increases exponentially with time until failure occurs.

## Swell

In solid mechanics, the terms **swell** and **rebound** are synonymous and refer to negative consolidation when a material is unloaded. In underground coal mining, the term 'swell' is ascribed a similar meaning in that it generally refers to the uplift of the immediate floor associated with the expansion of clay minerals in the presence of water. Montmorillonite is the most active of these minerals, increasing in volume by as much as 300 %. Research by Li et al. (2001) suggests that the roof and floor material immediately above and beneath a pillar are not prone to this behaviour because confinement prevents the ingress of moisture and resists swelling. However, material that is unconfined in an adjacent roadway may undergo substantial swelling. It is often difficult to distinguish the contribution that swelling makes to the overall uplift, or **heave**, of coal mine floor strata.

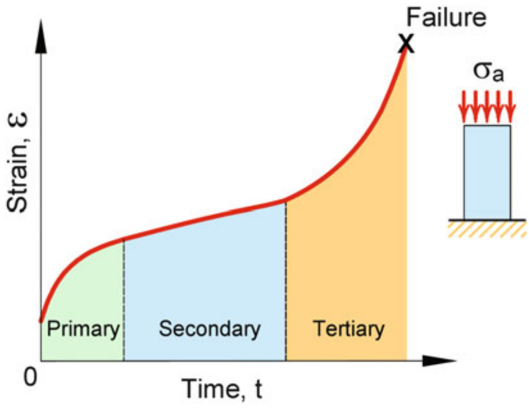

**Fig. A4.5** Stages of creep behaviour in rock

## Underground Experience with Applying Classical Bearing Capacity Principles

Experience to date in applying classical settlement and bearing capacity principles and design concepts to coal mining environments has demonstrated that there is significant uncertainty associated with these civil engineering approaches. A considerable amount of this experience derives from mining in Illinois in the USA and in the Lake Macquarie region of NSW, Australia. In the latter case, there has been a particular focus on the behaviour of the Awaba Tuff floor strata of the Great Northern Seam. This material is generically referred to as 'claystone' but mapping has shown it to be very variable in composition, thickness and strength, both vertically and laterally. It can comprise distinct layers of clay, shale, tuff, chert, sandstone and conglomerate to give a range in UCS from less than 0.5 MPa to well in excess of 40 MPa. Some tuffaceous layers have the potential to undergo a transition over time from a moderately hard rock to a saturated soil.

Table A4.4 summarises predicted versus measured outcomes as the result of applying settlement theory in partial pillar extraction panels in the Great Northern Seam at one mine (Mine A). The elastic settlement predictions were based on Eq. A4.5. The material was considered by the consultant to behave as an overconsolidated clay and so predictions of primary

**Table A4.4** Predicted versus measured elastic and primary settlement of claystone floor (Awaba Tuff) at a NSW mine.

| Mining panel | Predicted | | Measured settlement (mm) |
|---|---|---|---|
| | Elastic settlement (mm) | Primary consolidation settlement (mm) | |
| South P | 14 | 80 | 13 mm after 4 months and continuing |
| South J | 24 | 118 | 17 mm after 2.5 years and possibly stabilising |
| South M | 18 | 66 | 6 mm after 3 months, then stable |
| South O | 29 | 111 | 5 mm after 9 months |
| South West A1 | 25 | 86 | 17 mm after 12 months |

consolidation were also based on Eq. A4.5 using estimates of drained modulus.

Vasundhara et al. (1997) evaluated this predictive approach in a neighbouring mine (Mine B) where the Awaba Tuff was around 10 m thick and loaded by coal pillars that were 22 m wide by 29 m long. The material was treated as a normally consolidated clay with predictions being based on Eq. A4.6. Primary consolidation settlement of 1.7 m was predicted, which far exceeded measured surface subsidence over similar mine workings. As at day 836, the measured consolidation ranged from 30 mm to 140 mm, corresponding more closely to the outcomes at Mine A where analysis was based on only a 1 m thickness of material (Table A4.4). At both mine sites, the measured rate of consolidation differed markedly from the theoretical rate.

It is well known from civil engineering foundation practice that consolidation settlement estimates using consolidation test data directly can grossly over-estimate settlement for over-consolidated clays. The greater the over-consolidation, the greater the error in predictions. This comes about because of the effect of the vertical applied load on pore water pressures when there is lateral yield (Skempton and Bjerrum 1957). Hence, it might be expected that if applied to rock-like materials, Eq. A4.5 would result in a gross over-estimation of consolidation settlement, as appears may have been the situation in these case studies.

In civil engineering, it is common practice to apply a safety factor of around 2 to bearing capacity design, increasing to 3 where additional uncertainty exists in the adequacy and reliability of field data or the bearing capacity formulae, or where the consequences of failure would be severe. Application of bearing capacity formulae by Ganow (1975) to the back analysis of floor heave in the coal mines of Illinois led to the conclusion that failure was occurring at a safety factor of almost 7. Seedsman and Gordon (1992a) calculated a safety factor of 9.1 when the bearing capacity formula of Mandel and Salencon (1969) for infinitely long pillars was applied to rectangular and square pillars in the Great Northern Seam at Cooranbong Colliery. They concluded that this implied cohesion had to be 1/8 of the average laboratory value quoted by Seedsman and Mallet (1988a) and 1/13 of the value measured at the site.

Further application of the Mandel and Salencon (1969) formula by Seedsman and Gordon (1992a) to rectangular pillars of $L/w = 3.5$ in the Great Northern Seam at Cooranbong Colliery and $L/w = 3.2$ in the Fassifern Seam at Wyee Colliery produced safety factors of the order of 5 and 15, respectively. These safety factors were based on the most critical stability state of $\phi = 0°$. Seedsman and Gordon proposed an alternative model of bearing capacity failure as a means of avoiding unrealistic reductions in material properties whilst still employing classical bearing capacity formulae. This model was based on a number of assumptions, including that bearing capacity failure only occurred beneath the outer 1.5 to 2 m rib zone of a pillar. However, the model could not account for behaviour at Wyee Colliery.

Mills and Gale (1993) and Mills and Edwards (1997) studied 25 sites, of which 19 were located in the Great Northern Seam and five in the overlying Wallarah Seam in the Lake Macquarie region, with the other site being in Illinois, USA.

Significant surface subsidence had occurred at 16 of these sites, many of which comprised a partial extraction layout whereby every alternate row of pillars had been extracted. Mills and Gale (1993) reported that bearing capacity theory predicted failure loads that were four to five times greater than those at which these pillar systems typically failed. In order to produce bearing capacity failure loads that corresponded with pillar loads, the researchers had to equate the properties of the claystone floor to a fully saturated clay in an undrained state ($\phi = 0°$). They expressed the view that it was difficult to conceive claystone having a friction angle even as low as 20° and that it was their belief that the claystone material under the pillars retained essentially rock-like properties. The investigators went on to conclude that a classical bearing mechanism was not appropriate to explain the field behaviour, albeit that claystone behaviour was still involved in the pillar failure mechanism.

Assuming that classical settlement theory is applicable to Awaba Tuff in at least some circumstances, a range in laboratory derived values of 0.1 to 4.0 $m^2$/year for the consolidation index of this material translates to a 40 fold range in predictions of the time that it will undergo consolidation. Inaccuracies in drainage path length can be more critical because outcomes are proportional to the square of this factor. For the same circumstances, one investigator assumed drainage path length to be half the joint spacing in the claystone, whilst another assumed it to be half the thickness of the claystone bed. This translated to a 2500 fold difference in calculated consolidation time. A lack of data required Vasundhara (1999) to assume London Clay values for the compressibility index, $C_c$, and void ratio, $e_o$, of Awaba Tuff when calculating the primary consolidation settlement noted earlier.

A range of approximate methods have had to be applied to address the practicalities of determining material properties in a mining environment. Seedsman (2008) and Seedsman and Mallet (1988b) suggested a 50 % reduction in laboratory measured values of undrained and drained modulus for Awaba Tuff. Seedsman and Gordon (1992b) correlated uniaxial compressive strength and elastic modulus to moisture content. A ±50 % error band was associated with these correlations. Chugh and Pytel (1992) based estimates of the ultimate bearing capacity and deformation modulus of weak floor strata beneath a full size pillar on the results of bearing tests conducted with plates up to 0.6 m in diameter. The area of influence of plates of this size is very limited in comparison to the area of influence of a pillar footing.

Seedsman (2008, 2012) reported on the apparent successful application of the Mandel and Salencon bearing capacity formula to the design of partial extraction pillar systems at Awaba Colliery in NSW, a mine with a history of pillar foundation failure under stiff superincumbent strata. In 2014, however, surface subsidence in excess of one metre was detected over a portion of these workings.

The variety of assumptions and modifications associated with applying classical bearing capacity theory to underground coal environments and the range and accuracy of outcomes give rise to considerable uncertainty about the reliability of this approach. Most formulae are premised on laboratory scale testing, empirical models, and elastic and plastic theory concerned with the behaviour of soils, sands and clays assumed to be homogeneous to a depth of two to three times the width of the footing. Similar conditions rarely exist in underground coal mining environments. Therefore, application of bearing capacity formulae to foundation design in underground coal mining warrants careful risk assessment with consideration being given, in particular but not exclusively, to the following factors:

- The validity and accuracy of the formulae. There is a range of formulae and a variety of empirically derived bearing capacity factors. A range of values for shape factors has also been proposed in an attempt to adapt the two-dimensional solutions to three-dimensional circumstances. Laboratory scale studies involving sand and clay-like materials have featured strongly in the derivation of these factors and in formulae which address two layer situations. Each of these factors is a source of error in its own right. An arguably greater source of error arises when attempting to apply bearing capacity formulae to a mine environment, since the formulae are tailored to reasonably homogenous foundation materials and do not take account of variable material properties and defects, such as fractures and joints, inherent in coal pillar foundations.

- Material properties. It is well established in civil engineering practice that settlement and bearing capacity calculations are quite sensitive to the accuracy of input data and the reliable determination of the required material properties. There are serious practical, technological and financial limitations to sourcing appropriate, adequate and reliable data from underground coal mine environments, especially prior to mining having taken place.
- Dimensional scale. The width of a pillar footing can be an order of magnitude or more greater than typical civil engineering footings and, therefore, the zone of influence of the footing extends for a greater distance into the foundation. Consideration has to be given to whether the material within this zone of influence is homogenous and, therefore, whether it is valid to apply bearing capacity formulae in the given circumstances.
- Multiple layered situations. The wider footing and higher loading regime encountered in mining extends the zone of influence of the footing, thereby increasing the likelihood that foundation response will be affected by the behaviour of multiple layers of strata. It is likely that these multiple layers will contain a variety of materials, some with contrasting mechanical properties. Classical bearing capacity formulae do not explicitly account for these situations, although some formulae may find application to specific circumstances. Implicit approaches, such as calculating effective material properties of a number of layers and inputting these values into classical bearing capacity formulae, can fail to give proper consideration to specific layers modifying or controlling behaviour.
- Load scale. The loads to which pillar foundations are subjected are typically at least one order of magnitude greater than those encountered in civil engineering.
- Interaction between footings. The footing width to spacing ratio in a mining environment is inverse to that in a civil engineering environment, thereby giving rise to a much greater potential for interaction between footings.
- Footing construction. Pillar footings comprise natural, non-reinforced material that is embedded with vertical and horizontal defects (joints, cleat, bedding planes) and is of minimal tensile strength. Conversely, civil engineering footings are usually constructed of quality controlled, reinforced materials of higher tensile and compressive strength.
- Time. Rock mass properties can change over time in a mine environment, especially upon being exposed to moisture or sustained load.
- Flooding. Flooding of mine workings in time to come can have implications for, firstly, the selection of appropriate bearing capacity formulae at the design stage and, secondly, the range of material properties that the design procedure has to cater for over the required period of pillar system stability.

Unfortunately, it is a matter of fact that many of these factors cannot be adequately quantified, irrespective of which approach is adopted to foundation design.

## Conclusions

There have been a number of instances in NSW where elevated surface subsidence is very likely attributable to some form of classical bearing capacity failure of the pillar foundation and/or lateral plastic flow of material from beneath the pillars resulting in a change in the behaviour mode of the pillars. Induced tensile failure is not as likely to arise in civil engineering footings because there is a degree of control over site selection and footing design, including reinforcement against tensile failure.

Circumstances where very high safety factors need to be applied or material input properties severely modified in order to achieve bearing capacity predictions that are consistent with field performance suggest that the foundation behaviour mechanism or formula may not have been appropriately chosen; that this mechanism had deficiencies when applied to the particular mining environment; and/or that input material properties may not be representative of field properties. It is possible that on occasions, pillar foundation failure may have been the outcome of several interactive mechanisms, one of which may have involved classical foundation behaviour.

There is little doubt that some pillar system instabilities attributed to foundation failure would not have developed if the pillars had not been undersized for the function required of them and, therefore, overstressed in the first instance. It is also likely that some floor heave events associated with other mechanisms, such as swelling clay minerals or buckling failure of stiff beds in the floor, have been misconstrued as plastic flow and therefore, as classical bearing capacity failures.

Quantifying the type of foundation behaviour mechanisms operating in some mining environments is complicated further by the factor of time. The various bearing capacity formulae presented in Table A4.3 represent the worst-case situation of an undrained foundation, where $\phi = 0°$. Under normal circumstances, it is difficult to conceive the friction angle of most coal mine strata, including claystone, being less than 10°. Pillar load builds up over a period of time as the mining face is advanced, thereby providing time for some of the excess pore water pressure to be dissipated and for a partial recovery in friction angle. However, there is one circumstance which could give rise to friction angle being reduced to a very low value over an extensive area. This might occur when stiff, strong superincumbent strata fails and subsides over a very short time period so as to result in a regional step increase in pillar load.

Substantial interpanel pillars are one potential control measure for this hazard. They can be designed to limit panel span to prevent the stiffness of the superincumbent strata reducing to zero so that the panel pillars are not exposed to a rapid regional increase in load. They can also slow the rate of load transfer to the panel pillars, thereby providing time for the foundation material to drain and for friction angle to recover. In the worst case, interpanel pillars can prevent or retard the failure progressing through the mine. It is noteworthy that a significant number of failures attributed to floor bearing capacity in Australia have been associated with reducing the size of panel and interpanel pillars and with removing interpanel pillars.

Against this background, the process of settlement and bearing strength is still not fully understood and there may be other mechanisms involved that have yet to be identified or properly appreciated. Making design distinctions between foundation failure mechanisms and accounting for how interaction between coal pillar footings and foundation layers impacts on failure mechanisms is surrounded with uncertainty and warrants the assistance of numerical modelling. If civil engineering settlement and bearing capacity formulations are to be persevered with, it could be judicious to utilise numerical modelling to identify possible failure mechanisms and to undertake parametric and sensitivity studies to assess the vulnerability of design outcomes to uncertainties in input data and failure models.

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# Appendix 5: Formulae for Calculating Load on a Pillar Based on Abutment Angle Concept for the Most General Case

The formulations are based on the general case of parallelepiped pillars as defined in Fig. A5.1.

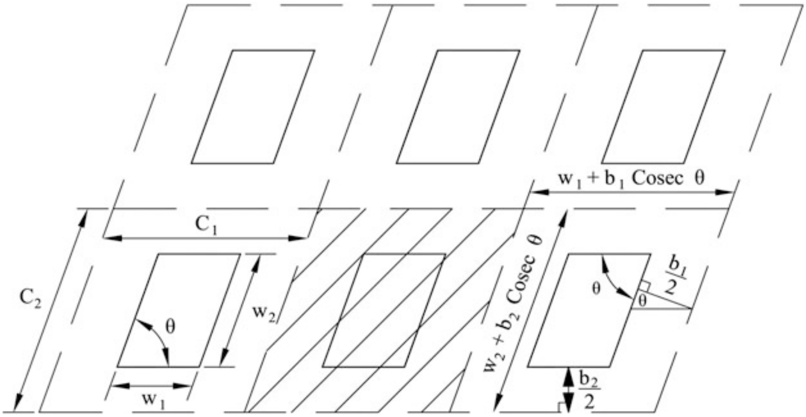
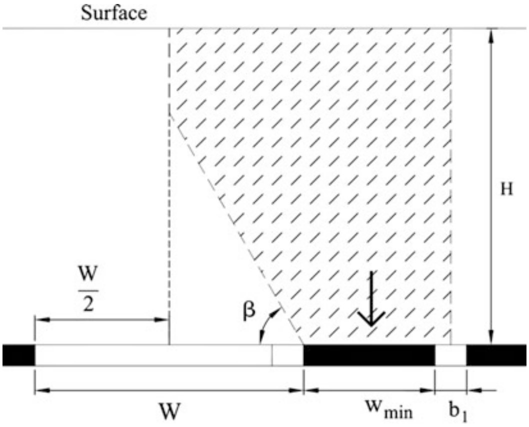

$w_1 + b_1$ Cosec θ
$C_1$
$C_2$
θ
$w_2$
$w_1$
$w_2 + b_2$ Cosec θ
θ
θ
$\frac{b_1}{2}$
$\frac{b_2}{2}$

**Fig. A5.1** Geometry associated with a layout of parallelepiped pillars

## Parallelepiped Pillars – Depth, $H \geq 0.5$ W Tan(β)


Surface
H
$\frac{W}{2}$
β
W
$w_{min}$
$b_1$

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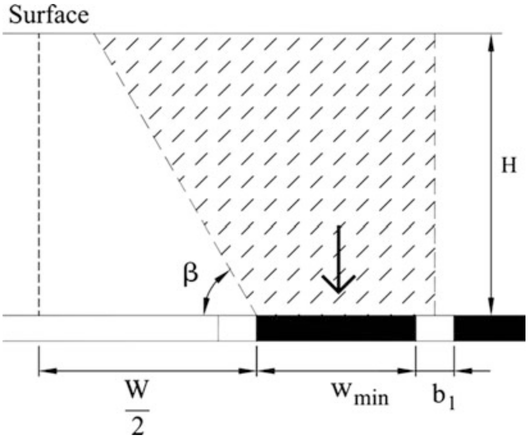
$$\sigma_{aps} = \frac{0.025\left[H\left(w_{\min} + \frac{b_1}{2} + \frac{W}{2}\right) - \frac{W^2 Tan\ (\beta)}{8}\right]\left[w_2 + \frac{b_2}{Sin\ (\theta)}\right]}{w_{\min}\, w_2}\quad (MPa) \tag{A5.1}$$

where:

$$w_{min} = w_1 Sin\ (\theta)$$

## Parallelepiped Pillars – Depth, H < 0.5 W Tan(β)



$$\sigma_{\text{aps}} = \frac{0.025H\left[w_{\min} + \frac{b_1}{2} + \frac{H}{2Tan\ (\beta)}\right]\left[w_2 + \frac{b_2}{Sin\ (\theta)}\right]}{w_{\min}\, w_2}\quad (MPa) \tag{A5.2}$$

where

$w_1$ = *pillar dimension parallel to cut-through*

$w_2$ = *pillar dimension parallel to goaf edge*

$\theta$ = *smaller internal angle between pillar sides ($\theta \leq 90°$)*

$w_{\min} = w_1$ *Sin*($\theta$)

$b_1$ = *bord width normal to goaf edge = gateroad width*

$b_2$ = *bord width normal to cut-through direction*

$W$ = *excavation span − rib to rib*

# Appendix 6: Timber Prop Performance Parameters

## Introduction

Specifications and recommendations for timber props vary with species and from country to country. A number of these are provided as points of reference.

## Australia

According to Menzies (c1970), Australian underground coal mining experience has shown that as a general rule, the diameter of a eucalypt hardwood prop should be one inch (25.4 mm) for every foot (0.3054 m) length of prop. This generates the metric prop dimensions presented in Table A6.1.

Shepherd and Lewandowski (1994) proposed that the strength of an Australian eucalyptus (hardwood) prop is given by:

$$P = 54.42 - 1.17\frac{L}{d} \tag{A6.1}$$

**Table A6.1** Australian hardwood prop dimensions based on recommendations of Menzies (c1970)

| Hardwood prop length (m) | Suggested prop diameter (mm) |
|---|---|
| 2 | 166 |
| 2.5 | 208 |
| 3 | 250 |
| 3.5 | 291 |
| 4 | 332 |

where

$P = load\,bearing\,capacity\,(tonnes)$
$L = prop\,length\,(mm)$
$d = prop\,diameter\,(mm)$

## South Africa

Figure A6.1 shows the load performance of 100 mm and 150 mm diameter wattle and saligna timber props plotted as a function of prop length.

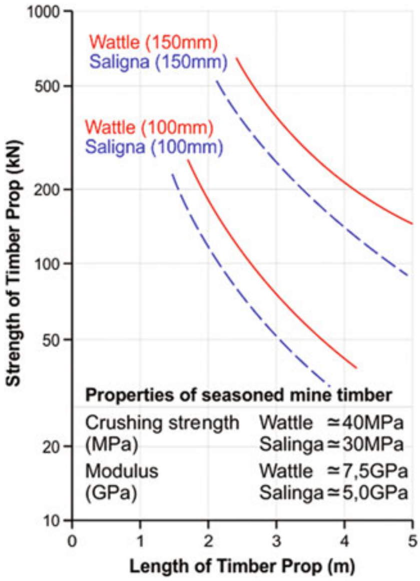


**Fig. A6.1** Strength properties of wattle and saligna timber props (Adapted from Wagner 1994)

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**Table A6.2** Kentucky State Government, USA, requirements pertaining to size of timber props and cross supports when used as the primary means of ground support in underground coal mines

| Post length (inches) | Diameter of round post (inches) | Cross-sectional area of split post (square inches) |
|---|---|---|
| 60 or less | 4 | 13 |
| Over 60–84 | 5 | 20 |
| Over 84–108 | 6 | 28 |
| Over 108–132 | 7 | 39 |
| Over 132–156 | 8 | 50 |
| Over 156–180 | 9 | 64 |
| Over 180–204 | 10 | 79 |
| Over 204–228 | 11 | 95 |
| Over 228 | 12 | 113 |

Note: 1 in. = 25.4 mm

## Kentucky, USA

Legislation in the USA state of Kentucky (Kentucky State Government 2013) requires that when timber props are used with cross supports as the primary means of support, the minimum diameter of round props shall be as recorded in Table A6.2.

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# Appendix 7: Standard Work Procedure for Setting a Timber Prop

## Standard Work Procedure

**Note**
This example of a Standard Work Procedure (SWP) for setting a timber prop in an underground coal mine should be risk assessed against site specific conditions and modified if required in order to achieve safe and effective outcomes.

## Requirements

1. Personnel
   (i) Minimum of 2 persons.
2. Training
   (i) Manual handling.
   (ii) Use of chain saw.
   (iii) Testing of roof and ribs.
3. Personnel Protective Equipment
   (i) Standard issue.
   (ii) Ear protection.
   (iii) Gloves.
   (iv) Safety glasses.
4. Tools
   (i) Sounding and barring down tool.
   (ii) Pointed nose shovel.
   (iii) Measuring stick.
   (iv) Chalk (white) or crayon (red).
   (v) Sledge hammer (6–7kg).
   (vi) Wood saw (bushman's hand saw or air chain saw).
   (vii) Temporary supports.
5. Materials
   (i) Timber props of adequate length.
   (ii) Timber end plates (foot plates/head plates/caps/lids).
   (iii) Timber wedges.
6. Inspections and Supervision
   (i) Workplace inspected by a mine official before persons start work.
   (ii) Workplace inspected by persons setting prop before starting work and regularly during work.
   (iii) Instructions issued as to where props are to be installed.

## Procedure

1. Verify that the work place is safe.
   (i) Check that area has been inspected by a supervisor and declared safe for persons to enter.
   (ii) Visually check area from a safe distance.
   (iii) Listen for sounds of movement and instability.
   (iv) Sound roof from a safe location. A safe distance is based on not being in line-of-fire if the roof falls while being sounded.

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    (v) Use a timber-based tool for sounding coal roof and a steel-based tool for sounding stone roof. A dull thud indicates that the immediate roof is parting from the rock mass.
    (vi) If temporary support is required, it must be set without venturing out under strata that is not secure.
2. Look for a niche or ridge in the roof at or close to the target site.
    (i) The niche has to have an open side so that the hammer can hit the wedge clean and not bounce off the roof first. Preferable roof feature is a small ripple or ridge.
    (ii) If setting a prop beneath a keystone, near enough is not good enough.
3. Mark prop location if it is not readily identifiable, especially if other persons are to set the prop.
    (i) Use a chalk or red crayon cross on the roof or, preferably, a chalk cross or number on a wooden wedge placed on the floor where the base of the prop is to be located.
    (ii) Experienced miners will often mark the measuring stick with a corresponding number at the required length of the prop.
4. Clear down to a solid coal or floor, preferably level, beneath niche or ridge.
    (i) Location can be determined by dropping an identifiable piece of stone or coal from the roof location to the floor.
    (ii) If a foot plate is being used, ensure area cleared is large enough to accommodate the foot plate.
5. Set foot plate if required.
6. Measure floor to roof.
    (i) If using a foot plate, place it before measuring.
    (ii) Remember to allow for any head plate.
    (iii) Use a measuring stick and make sure it has clearance out of any roof niche (to verify prop can be placed into niche).
    (iv) When setting more than one prop at a time, mark the prop number on the stick after each measurement is taken.
    (v) Any prop that has a diameter greater than 140 mm should be cut an extra 10 mm short to provide the clearance to expedite its setting and to accommodate the wedge.
7. Cut prop to length.
    (i) Cut props from an elevated position. This may be in a timber pod, over a tripod, or by using one prop as a support rest for another.
    (ii) Use a bushman’s saw if not trained in the use of a chain saw.
    (iii) Prop ends should be cut square. This may require cutting off both ends. Otherwise cut off the end with the smallest diameter.
8. Subject to prop weight and physical conditions, two or more persons may be required to manually handle the prop.
    (i) Where more than one person is required they must lift together and both carry from the same side.
    (ii) At the setting location, one person is to place the larger end of the prop in the correct floor position and then both persons are to lift the prop up to its vertical position.
9. Manoeuvre top of prop into niche or up against ridge.
10. Place wedge into position from a direction that results in prop being restrained by niche or ridge as the wedge is being hammered home.
11. Tap wedge with hammer until it starts to bite into prop.
12. Verify that prop is vertical. Eyeball from front and side.
13. Set wedge tight, taking care to continue to keep prop vertical.

Variations

- If roof is uniform or a head plate is being used, the prop may need to be periodically tapped back into a vertical position as the wedge is being set.
- If a second person is used to steady the prop until the wedge is set, that person must position themself out of the line of fire of the person setting the wedge. All other persons must keep well clear of the hammer swinger.
- In high workings, where the top of the prop cannot be reached or where the wedge cannot be set in an ergonomic manner, the wedge may need to be set at the base of the prop. A clear access needs to be provided for the hammer to be able to hit the wedge.
- If a head plate is required in high workings, nail it to the prop prior to lifting the prop into position.

## Completion of Work

1. Place all off-cuts in timber pod for removal from mine or in a place where they do not present a trip hazard.
2. Collect all tools and place with remaining timber props.
3. Document/report number and location of props set.
4. Document/report any hazards encountered during the work.
5. Pass on documents/reports to supervising mine official.

# Appendix 8: Derivation of Geometric Relationship for Deflection of a Chord

## Mathematical Derivation

The geometry and trigonometry associated with the formulation of the concept of mechanical advantage is shown in Fig. A8.1. The vertical column (beam) $A_0$–C is assumed to be pinned (hinged) at both ends and to deflect in a perfect arc, with the end points of the line (or roof beam abutments) moving inwards with increased deflection (or curvature) of the line. Hence, the length of the line, or roof beam, remains constant. The amount of inward deflection of the abutments under the effect of a lateral load, P, is designated '$U_{hi}$', with the corresponding mid-point line (beam) deflection designated '$U_{vi}$'.

Revised derivation of so-called Mechanical Advantage

Based on Fig. A8.1, the formula for so-called 'mechanical advantage' at the i^th increment of displacement, $U_{vi}/U_{hi}$, is given by Eq. A8.1.

$$\frac{U_{vi}}{U_{hi}} = \frac{1 - Cos\left(\frac{\theta_i}{2}\right)}{\theta_i - 2Sin\left(\frac{\theta_i}{2}\right)} \tag{A8.1}$$

$$Let\, \alpha_i = \frac{\theta_i}{2}$$

$$Therefore,\ \frac{U_{vi}}{U_{hi}} = \frac{1 - Cos(\alpha_i)}{2(\alpha_i - 2Sin\,(\alpha_i))}$$

$$Now,\ Cos(\alpha_i) = 1 - \frac{\alpha_i^2}{2!} + \frac{\alpha_i^4}{4!} - \ldots$$

$$and\, Sin\,(\alpha_i) = \alpha_i - \frac{\alpha_i^3}{3!} + \frac{\alpha_i^5}{5!} - \ldots$$

If $\alpha_i$ is small, say less than 0.1 radians, then to sufficient accuracy for engineering purposes:

$$Cos(\alpha_i) = 1 - \frac{\alpha_i^2}{2!}$$

$$Sin\,(\alpha_i) = \alpha_i - \frac{\alpha_i^3}{3!}$$

$$\text{so}\,\frac{U_{vi}}{U_{hi}} = \frac{1 - \left[1 - \frac{\alpha_i^2}{2}\right]}{2\left(\alpha_i - \left[\alpha_i - \frac{\alpha_i^3}{6}\right]\right)}$$

Therefore

$$\frac{U_{vi}}{U_{hi}} = \frac{\frac{\alpha_i^2}{2}}{\frac{\alpha_i^3}{3}} = \frac{3}{2\alpha_i} = \frac{3}{\theta_1} \tag{A8.2}$$

Now, from Fig. A8.1:

$$\frac{U_{vi}}{L} = \frac{\left[1 - Cos\left(\frac{\theta_i}{2}\right)\right]}{\theta_i}$$

To substitute for $\theta_i$ in Eq. A8.2 in terms of $U_{vi}$, rearrange the formula and write it instead in terms of $\alpha_i$.

Therefore

$$U_{vi} = \frac{L}{2\alpha_i}[1 - Cos(\alpha_i)] = \frac{L}{2\alpha_i}\left[1 - \left(1 - \frac{\alpha_i^2}{2}\right)\right]$$

$$= \frac{L}{2\alpha_i} \cdot \frac{\alpha_i^2}{2} = \frac{L\alpha_i}{4} = \frac{L\theta_i}{8}$$

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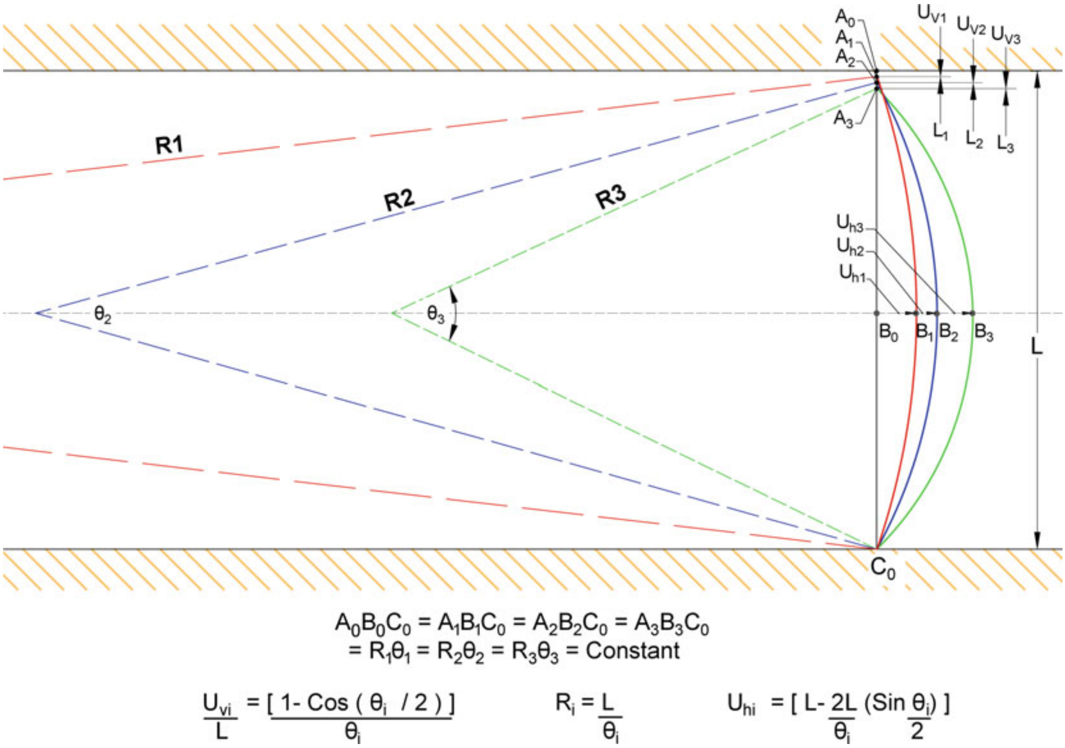
$A_0$
$A_1$
$A_2$
$A_3$
$U_{V1}$
$U_{V2}$
$U_{V3}$
$L_1$
$L_2$
$L_3$
R1
R2
R3
$\theta_2$
$\theta_3$
$U_{h3}$
$U_{h2}$
$U_{h1}$
$B_0$
$B_1$
$B_2$
$B_3$
L
$C_0$
$A_0B_0C_0 = A_1B_1C_0 = A_2B_2C_0 = A_3B_3C_0$
$= R_1\theta_1 = R_2\theta_2 = R_3\theta_3 =$ Constant
$\frac{U_{vi}}{L} = \frac{[1 - \mathrm{Cos}(\theta_i / 2)]}{\theta_i}$
$R_i = \frac{L}{\theta_i}$
$U_{hi} = [L - \frac{2L}{\theta_i}(\mathrm{Sin}\frac{\theta_i}{2})]$

**Fig. A8.1** Geometry and revised trigonometry to more accurately reflect the mechanics and mathematics associated with the concept of mechanical advantage presented by Frith (2000) and Colwell (2004, 2012)

Rearranging this result

$$\theta_i = \frac{8U_{vi}}{3L}$$

and substituting this into Eq. A2.2 gives:

$$\frac{U_{vi}}{U_{hi}} = \frac{3L}{8U_{vi}} \tag{A8.3}$$

Therefore

$$U_{hi} = \frac{8(U_{vi})^2}{3L} \tag{A8.4}$$

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Colwell, M. (2004). *Analysis and design of rib support (ADRS). A rib support design methodology for Australian Collieries* (p. 325). ACARP end of project report C11027. Brisbane: Australian Coal Association Research Program (ACARP).

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Frith, R. (2000). *The use of cribless tailgates in longwall extraction* (pp. 84–92). Paper presented at the 19th international conference in ground control in mining, Morgantown, WV. West Virginia University.

# Appendix 9: Three Major Incidents in Australia Related to the Design of Pillar Extraction Panels

## Some Major Pillar Extraction Incidents in Australia

### Moura No. 4

#### Introduction

On 16th July 1986, an explosion occurred in a pillar extraction section at Moura No. 4 Colliery in Queensland, Australia, claiming the lives of the 12 miners in the section. The inquiry found that a roof fall had occurred in the goaf and that the windblast from the fall blew a mixture of methane, air and coal dust into the working area. This was ignited, with some eight possible sources of ignition being investigated. The inquiry considered that a flame safety lamp, although properly assembled, was the most likely source of ignition. However, many remain convinced that the source was frictional ignition due to rock on rock contact during caving of the immediate sandstone roof.

#### Overview

This overview is based on the findings of the Warden's Inquiry into the incident (Lynn et al. 1987). Figure A9.1 shows the mine plan associated with the site of the incident. The first plan of extraction called for a single split through the formed pillars and extraction of bottom coal to the base of the seam. Thickening of a stone band in the middle of the seam caused a major decrease in the quality of the coal mined by this method, causing management to modify the method of extraction.

The second plan of extraction called for two splits to be driven through the 50 m long by 30 m wide pillars to a height of 2.3 m and then extracting bottom coal before partially extracting the fenders. This method proved unsuccessful. Irregular sized stooks were left which neither fully supported the roof nor permitted the development of full goaf caving. As a result, abutment pressures increased, as evidenced by rib spall and floor heave. Concern over the method of extraction was expressed by the District Union Inspector on 5th June 1986.

In the meantime, the company acquired the surface property over the mining panel. This removed a major constraint on not causing surface subsidence. With weighting apparent in the panel, management decided on a system of total extraction of the formed pillars. It was decided that:

- extraction would be carried out on a two shift basis;
- three splits would be driven through the pillars and the fenders so formed totally extracted; and
- progress would be closely monitored during the trial period of 2 weeks.

Total extraction continued without major problems until 16th July, being the date of the

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explosion. Observations made by management during an inspection on 15th July confirmed a belief that a goaf fall would shortly occur in the area of total extraction and that this would relieve the abutment loadings being experienced by the pillars. On the morning of the incident, the Undermanager inspected the goaf edge, observed 'nipping' of the props and believed a fall was going to occur on that shift.

The expected goaf fall occurred soon thereafter, with the windblast from the fall blowing a mixture of methane, air and coal dust from the goaf and inbye working area out to No. 26 - cut-through and perhaps beyond. An explosion occurred which caused extensive damage inbye of No. 22 cut-through and the loss of 12 lives.

The inquiry found that neither of the first two plans for partial pillar extraction was strictly followed and substantially smaller remnant pillars were left than planned. It recommended research and experimentation into the phenomenon of frictional ignition with the purpose of establishing a standard whereby all strata rock found in Queensland could be classified according to their degree of incendivity. Furthermore, it recommended that pillar extraction be permitted only with approval of the Mines Department Inspectorate, who should require that:

- no departure from the approved method and sequence of extraction be undertaken unless of a minor nature and then only with the specific approval of the Mine Manager;
- a plan of the method and sequence for extracting pillars be displayed on notice boards on the surface of the mine and in the working panel; and
- in the case of a partial extraction method, adequate control to ensure that remnant pillars are not extracted beyond their final design dimensions.

## Moura No. 2 Colliery, Australia

### Introduction

On 7th August 1994, an explosion occurred in a partial pillar extraction section at Moura No. 2 Colliery in Queensland, Australia, claiming the lives of the 11 miners in various parts of the mine. The inquiry found that the explosion was caused by spontaneous combustion in 512 Panel igniting a flammable mixture of gas as it was passing through the explosive range following sealing of the panel a few days earlier.

### Overview

This overview is based on the findings of the Warden's Inquiry into the incident (Windridge et al. 1996). A plan of 512 Panel is shown in Fig. A9.1. The mine was noted to be gassy and prone to spontaneous combustion and 512 Panel had been predrained. Prior to about 1986, panels had been designed for the goaf to collapse during pillar extraction but more recent designs, including that of 512 Panel, were for the goaf to remain open and be supported by leaving selected pillars either totally or partially in place. It was believed that an open and ventilated goaf would mitigate the risk of spontaneous combustion.

Strata control with the need for regional stability was a dominant consideration in the design of 512 Panel. The panel was divided into three compartments of roughly equal size, separated by two rows of large compartment pillars disposed across the panel (Fig. A9.2). The headings through these large pillars did not all align with those through the smaller pillars within each compartment. The most inbye cut-through (No. 13) and the top return airway were kept open for ventilation and inspection purposes.

The development of 512 Panel commenced in November 1993 and comprised 7.5 m wide headings and cross-cuts to form the layout of pillars described. The first workings were limited in height to the top 3 m of the seam with the intention to mine to the full seam height during pillar extraction.

The extraction phase commenced in April 1994 and involved rib-stripping alternative rows of pillars within each compartment so as to leave narrow L-shaped stooks between adjacent rows of intact pillars. This was, in effect, a 'take a row, leave a row' method of pillar extraction. The extraction phase also involved the systematic mining of approximately 1.5 m of bottom coal by ramping down in the exposed coal floor to the

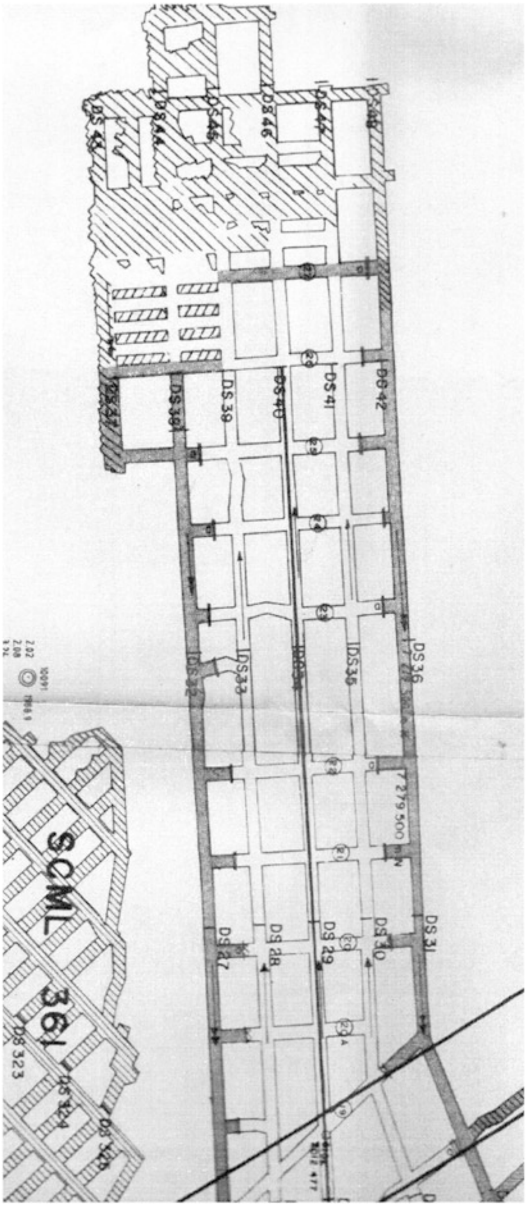

**Fig. A9.1** Plan of Moura No. 4 incident site (after Lynn et al. 1987) © State of Queensland. Department of Natural Resources and Mines 2013

base of the seam with the remote controlled continuous miner.

The inquiry determined that:

- The failure to prevent the development of a heating in 512 Panel was attributed to a number of aspects of the design and operation of the panel together with certain beliefs concerning panel life in relation to an assumed spontaneous combustion incubation period.
- While no one feature of the design and operation of 512 Panel was identified as directly causing the development of a heating, a number were considered less than desirable in that respect.
- The first of these was the amount of loose coal left from the mining process, fracturing of the ribs and stook instability. Both of these were considered undesirable from the point of view of spontaneous combustion management.
- While loose coal is inevitably left with any method of extraction, the particular way bottom coal was extracted in 512 Panel by ramping down probably left greater quantities than had been the case with other methods of extraction.
- This situation was worsened by limiting the length of ramps since a certain quantity of cut, but unrecovered, coal was left in each ramp. These significant quantities of loose coal in the ramp areas would likely not be effectively ventilated. This may have been exacerbated by local roof falls burying the unrecovered coal.
- In addition, high ribs adjacent to the ramp areas were prone to collapse, giving rise to accumulation of loose coal at the sides of stooks and pillars. The stresses induced on remnant pillars would have been sufficient to cause some fracturing of the coal giving rise to the potential for the ingress of air and so the development of a deep seated heating.
- While a relatively high ventilation quantity was available in the 512 Panel it is very likely that, because of large open areas, ventilation was somewhat sluggish in the goaf. Although sluggish ventilation may well have been adequate to effectively ventilate the goaf, if the intent of the panel ventilation design had been adhered to and had other factors not intruded. Those other factors, however, caused undesirable (from the point of view of the prevention of a heating) loss of, or variation in, ventilation to parts of the goaf.

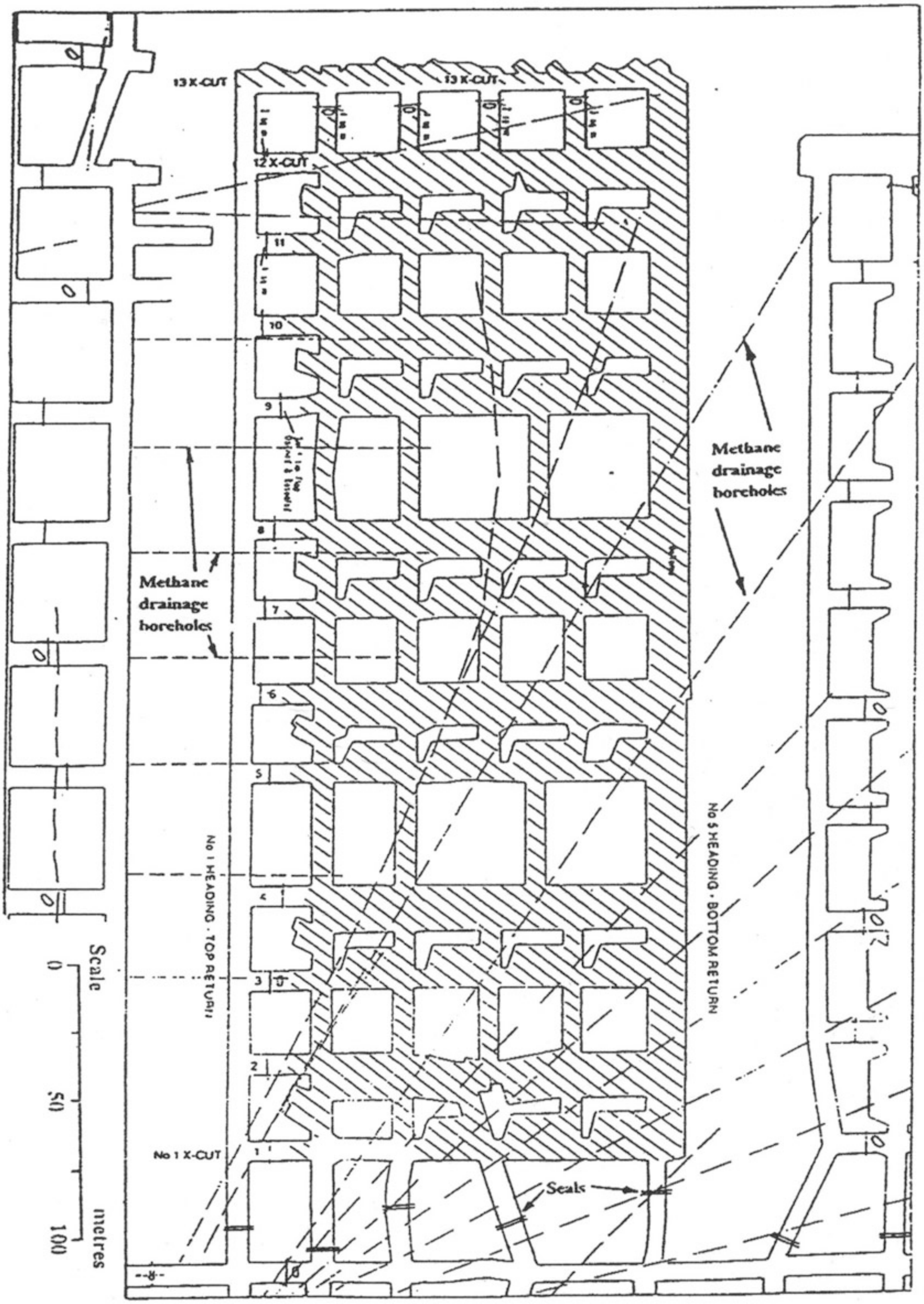
13 X-CUT
13 X-CUT
12 X-CUT
11
10
9
8
7
6
5
4
3
2
1
No 1 X-CUT
Methane drainage boreholes
Methane drainage boreholes
No 1 HEADING · TOP RETURN
No 5 HEADING · BOTTOM RETURN
Seals
Scale
0
50
100
metres

**Fig. A9.2** Plan of Moura No. 2 incident site (After Windridge et al. 1996) © State of Queensland. Department of Natural Resources and Mines 2013

- The intent of the ventilation design was that holes in the stoppings between the back row of pillars would act, in effect, as regulators to balance the ventilation across all parts of the goaf. In practice, however, there was evidence that these were affected by roof falls or local strata instability and that their function was, at times, compromised.
- There would inevitably be areas in the goaf which were likely to have been less than effectively ventilated, notably in cross cuts between headings and in the corner of the L shaped remnant pillars. These would have been fertile areas for the development of a heating.

The inquiry recommended that legislation be amended to make it a requirement that spontaneous combustion be specifically included as a factor to be considered and evaluated when applying for approval to conduct secondary extraction.

## Endeavour Colliery, Australia

### Introduction

On the 28 June 1995, an explosion occurred in 300 pillar extraction Panel at Endeavour Colliery in NSW Australia. There were 30 miners underground at the time, including 8 in the pillar extraction crew who, despite varying injuries and disorientation arising from lack of visibility, found their way out of the panel. Investigations into the incident found that a goaf fall had expelled a flammable mixture of gas into the workplace, where it was ignited by a likely but unconfirmed source, being a shuttle car cable back-to-back electrical connector.

### Overview

This overview is based on the findings of an investigation into the incident detailed in Mine Design Guideline MDG 1007 produced by the NSW Government (MDG-1007 1996). The mine plan associated with the site of the incident is shown in Fig. A9.3. The investigation reported that:

- Although some extraction panels may have been ventilated without bleeders, it was the most common practice at Endeavour Colliery to ventilate goaf areas with bleeder returns.
- The pillar extraction area worked prior to 300 Panel was ventilated ‘over and through the goaf’.
- When 300 Panel was designed, there was an intention to form a bleeder return by holing into the Buff Headings. This did not prove possible due to the Buff Headings subsequently being used to store water pending the installation and approval of surface pumping outlets.
- Poor roof conditions prevented an attempt to skirt around the water accumulation.

Conclusions included:

- There was a failure of panel design, development and review processes to recognise and effectively treat the hazard presented by the potential for accumulation of gas in the 300 Panel goaf and the possibility of windblast.
- There was a failure of persons on-shift and, in particular, those in positions of supervision and control, to recognise the potential hazard represented by occurrences of gas and an imminent large fall of roof in the goaf.

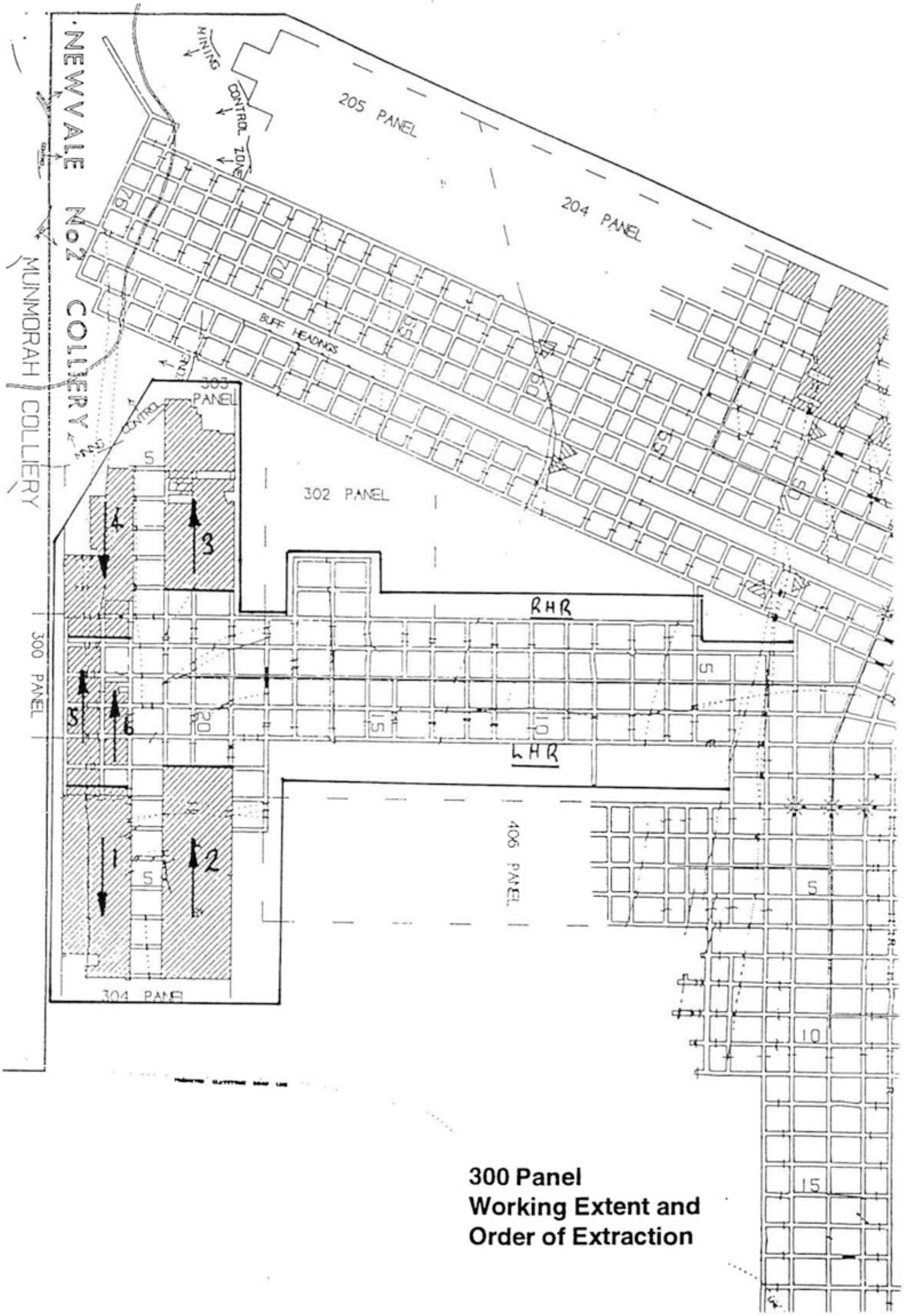
NEWVALE No2 COLLIERY
MUNMORAH COLLIERY
MINING CONTROL ZONE
MINING CONTROL ZONE
205 PANEL
204 PANEL
BUFF HEADINGS
303 PANEL
302 PANEL
RHR
LHR
300 PANEL
406 PANEL
304 PANEL
1
2
3
4
5
6
5
10
15
20
300 Panel
Working Extent and
Order of Extraction

**Fig. A9.3** Plan of Endeavour Colliery incident site (from MDG-1007 1996)

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# Appendix 10: Advantages, Disadvantages and Operational Aspects Relating to Mobile Roof Supports

## Mobile Roof Supports (MRS)

### Advantages and Disadvantages

Advantages of MRS

- Reduction in manual handling injuries due to, typically, a 90 % reduction in timber prop usage.
- Remove the need for persons to work at an active goaf edge.
- Remotely controlled, removing operators back from the face line and out of intersections.
- Provide a higher support density and level of roof coverage than timber props.
- Quicker to set than timber.
- Can be set under unsupported roof that would otherwise be out of bounds when using timber breaker props and rock bolt breaker lines – particularly applicable when 3 MRS are used to lift left and right.
- Improved stability. Unlike timber props, cannot be displaced by roof or rib falls and capable of handling eccentric loads and lateral displacements.
- Positive set load.
- Capability to yield.
- Can be repositioned to suit circumstances.
- Accelerate the mining cycle, thus providing less time for ground conditions to deteriorate.
- Capacity to extract coal in roof conditions that would be abandoned if using timber due to reduced support capacity and density and exposure to risk in setting timber.
- Increase the rate of pillar extraction and productivity.

Disadvantages of MRS

- May require a proven system of extraction to be compromised by changing dimensions or sequences in order to incorporate MRS.
- Loss of roof monitoring and warning of impending goaf fall provided by timber props.
- Susceptible to damage in massive roof conditions, especially when delayed caving occurs.
- Vulnerable to being damaged by the continuous miner.
- Introduces additional risks associated with cable handling and recovery of a MRS immobilised under unsupported roof or trapped by a fall of ground.
- May cause operations to stop in the event of a breakdown.
- Operators can become over-confident and complacent, acquiring a false sense of security. In three of the five fatalities in MRS sections to 2006 in Kentucky, USA, the deceased were standing next to the MRS units in the active intersection after extracting the last pushout stump, or stook (Marshall Miller and Associates 2006).

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## Operational Practices

Operational practices should be detailed in a Standard Work Procedure (SWP) that has been developed with operator input and subjected to risk assessment. Regard may be had to the following guidance material and features of many SWPs relating to the use of MRS units in pillar extractions:

- Guidelines:
  - MDG 1005: Manual on Pillar Extraction in NSW Underground Coal Mines (MDG-1005 1992).
  - MDG 5002C: Guideline for the Use of Remote Controlled Mining Equipment in Underground Coalmines (MDG-5002C 2011).
- Manual control should only be used for maintenance purposes.
- During lifting:
  - MRS units should be kept as close as practical to the solid abutment without impeding the operation of the continuous miner.
  - MRS units should be kept at least 0.3 m apart to avoid becoming interlocked.
  - The number of persons working in the vicinity of the continuous miner and the MRS units should be kept to a minimum and their designated standing positions clearly identified.
- At all times, it is good practice to stay well back (preferably, at least 6 m) from a MRS but particularly when it is being pressurised or depressurised. Operators need to be made aware that the setting of a MRS can cause failure of immediate roof skin or beam.
- When setting a MRS to the roof, set the front legs first to deflect any broken bolt heads away from the workplace.
- When lowering a MRS, lower the rear legs first to allow any debris to fall to the rear of the machine so it does not impede tramming of the unit.
- Clean up loose material in front of the units using the continuous miner.
- MRS units in an active mining area should be advanced in a leap frog manner, in intervals not exceeding half a canopy length (and less in poor ground conditions).
- Procedures should be pre-planned and risk assessed for dealing with situations where units breakdown or become immobilised or buried.
- When recovering a partially or totally buried MRS unit, the canopy should not be lowered until alternative supports have been set to prevent the goaf flushing in further.

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# Appendix 11: A Selection of Design Requirements and Guidelines Relating to Controlling Surface and Aquifer Water Inflow

**Table A11.1** Examples of regulations, guidelines and practices relating to extraction under water bodies and aquifers (extended and modified from Byrnes 1999)

| Country | Minimum cover | Minimum carboniferous strata thickness | Target maximum tensile strain | Design conditions | References |
|---|---|---|---|---|---|
| Australia | 40 m | – | | Up to 50 % under the sea extraction by bord and pillar. | Kapp and Williams (1972) |
| | 65 m | – | | Bord and pillar beneath water storage dams. | |
| | Bord and Pillar – 46 m of solid rock head | – | | First workings – no caving | Wardell (1975) |
| | | | | Pillar size as prescribed by legislation for mining under land | |
| | Panel and Pillar partial extraction – 46 m of solid rock head | – | 7.5 mm/m | Partial extraction<br>- The maximum width of any totally extracted panel shall not exceed 0.3 times the solid rock head cover thickness.<br>- The minimum width of any abutment pillar between extraction panels shall be 8 times its height or 0.12 times the solid rock head cover, whichever is greatest.<br>- No total extraction or pillar extraction permitted within a distance of 46 m of any known fault having a vertical displacement >3 m or dyke having a width >6 m | |
| | Longwall– 46 m of solid rock head and 60 m of solid rock head cover for each metre of extracted height | – | 7.5 mm/m | No total extraction or pillar extraction permitted within a distance of 46 m of any known fault having a vertical displacement >3 m or dyke having a width >6 m | |

(continued)

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**Table A11.1** (continued)

| Country | Minimum cover | Minimum carboniferous strata thickness | Target maximum tensile strain | Design conditions | References |
|---|---|---|---|---|---|
| Austria | 290 m | – | – | Total extraction | Byrnes (1999) |
| Canada (Nova Scotia) | 213 m | – | 6 mm/m | Total extraction | Garritty (1983) |
| | 213 m and 100 m of solid rock head cover for each metre of extracted height | | 7.7 mm/m | Total extraction | Kapp and Williams (1972)<br>Byrnes (1999) |
| Canada (Vancouver Island) | 122 m | | | Total extraction<br>Extracted thickness = 1.5 m | Byrnes (1999) |
| Chile | 150 m | | 5 mm/m | Total extraction | Kapp and Williams (1972)<br>Garritty (1983) |
| India | – | – | 3 mm/m | Total Extraction<br>A higher strain limit may be considered if superincumbent strata >35 % shale | Saxena and Singh (1982) |
| Japan | 60 m | | 8 mm/m | Maximum mining height at minimum rock head cover = 0.8 m | Garritty (1983) |
| Turkey | 160 m | – | – | – | Birön, 1964 |
| UK | Panel and Pillar – 60 m | 45 m | 10 mm/m | Partial extraction | NCB (1968)<br>NCB (1971) |
| | Longwall – to seabed – 105 m | 60 m | 10 mm/m | Maximum mining height at minimum rock head cover = 1.7 m | NCB (1968)<br>NCB (1971) |
| | Longwall – to base of aquifer with inrush potential – 45 m | – | – | – | NCB (1975)<br>Look at legislation and guideline |
| USA | 60 ft for each foot of extraction height (~60m/m) | – | 8.75 mm/m | Total extraction | Babcock and Hooker (1977) |
| | 60 m of solid rock head cover for each metre of extracted height | – | 8.75 mm/m | Total extraction | Byrnes (1999) |
| Yugoslavia | 108 m | – | | – | Byrnes (1999) |

**Table A11.2** A selection of proposals for the thickness of zones associated with hydrogeological models (extended and modified from Forster 1995)

| | | Thickness of deformation | | | | |
|---|---|---|---|---|---|---|
| Author | Country | Caved zone | Fractured zone$^{+}$ | Constrained zone* | Surface zone | Remarks |
| Kenny (1969) | UK | 2–4 t | – | – | – | Caving observations |
| Silitsa and Vasilenko (1969) | USSR | – | 16 t | 4 t | – | Measured results using dyes |
| Kapp and Williams (1972) | Australia | – | <30 m (assumed by authors to be 100 ft) | – | 15 m (50 ft – adopted by authors from Orchard, 1969) | Zone thicknesses are only estimates (assumptions) |
| Ropski and Lama (1973) | Poland | 1.5–2 t | 3–3.5 t | – | – | Borehole observations |
| Orchard (1974) | UK | 6–9 m | 18–36 m | 30 m | <30 m | Constrained zone should contain 50 % shale |
| Morton (1975, 1976) | Australia (Wongawilli and Kemira) | <30 m | 34 t (94 m) | – | 12 m | Based on measurements of permeability and piezometric pressure |
| Wardell (1975) | Australia (Newcastle) NSW | <5 t | <10 t | 50 t-S | S | S = assumed surface zone thickness |
| Wardell (1973, 1976) | Australia (NSW South Coast) | | 10 t | 60 m | 12–15 m | Recommendations based on overseas experience |
| Singh & Kendorski (1981) | UK | 3–6 t | 30–58 t | 9–27 m | <15 m | Constrained zone thickness depend on lithology |
| Holla and Armstrong (1986) | Australia (NSW Hunter Valley) | 2 t | 10–13 t (36–45 m) | – | – | Borehole anchors used to determine strains |
| Singh (1986) | UK | 3 t | 100t/(3.1t + 5) | 8 t | <15 m | Weak strata |
| | | 5 t | 100t/(1.2t + 2) | 15 t | 15 m | Strong strata |
| Kesseru (1984) | Hungary | – | 20–40 m | 15–25 m | – | Field experience |
| Kesseru et al. (1987) | | | | | | |

*t* seam thickness *includes caved zone thickness + recommended safe distance for subaqueous mining

**Table A11.3** Summary of recommendations concerning strata that comprises the constrained zone (after Forster and Enever 1991)

| Author | Remarks |
|---|---|
| Babcock and Hooker (1977) | Solid strata with no alluvial or marine sediments |
| Orchard (1974) | Should contain 50 % impermeable shale and siltstone |
| Wardell (1975) | Should contain beds of naturally low permeability |
| Reynolds (1976) | Narrabeen Group rocks (NSW South Coast) |
| Singh and Kendorski (1981) | Should contain at least one bed with low permeability |
| Garrity (1983) | Cover depth >100 m, at least one third of which should be sandstone (for competence), no faults – based on observations in mines |
| Bhattacharya and Gurtunca (1984) | Should contain at least one bed with low permeability, no open cracks or fissures, no faults or other weakness planes |
| Kesseru et al. (1987) | Should contain impermeable beds with $\sigma_{min} > \sigma_w$ ($\sigma_w$ = piezometric pressure) |
| Kesseru (1988) | |

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# Appendix 12: A Selection of Classification Schemes Relating to Subsidence Impacts on Structures

**Table A12.1** National Coal Board system for classifying subsidence damage

| Change in Length of Structure | Class of Damage | Description of Typical Damage |
|---|---|---|
| Up to 0.03 m | 1. Very slight or negligible | Hair cracks in plaster. Perhaps isolated slight fracture in the building, not visible on outside. |
| 0.03 - 0.06 m | 2. Slight | Several slight fractures showing inside the building. Doors and windows may stick slightly. Repairs to decoration probably necessary. |
| 0.06 - 0.12 m | 3. Appreciable | Slight fracture showing on outside of building (or one main fracture). Doors and windows sticking. Service pipes may fracture. |
| 0.12 - 0.18 m | 4. Severe | Service pipes disrupted. Open fractures requiring rebonding and allowing weather into structure. Window and door frames distorted. Floors sloping noticeably. Walls leaning or bulging noticeably. Some loss of bearing in beams. If compressive damage, overlapping of roof joints and lifting of brickwork with open horizontal fractures. |
| More than 0.18 m | 5. Very Severe | As above, but worse, and requiring partial or complete rebuilding. Roof and floor beams lose bearing and need shoring up. Windows broken with distortion. Severe slopes on floors. If compressive damage, severe buckling and bulging of roof and walls |

Adapted from NCB (1975)

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**Fig. A12.1** National Coal Board system for classifying subsidence impact based on relationship between the length of a structure and horizontal ground strain (Adapted from NCB (1975))

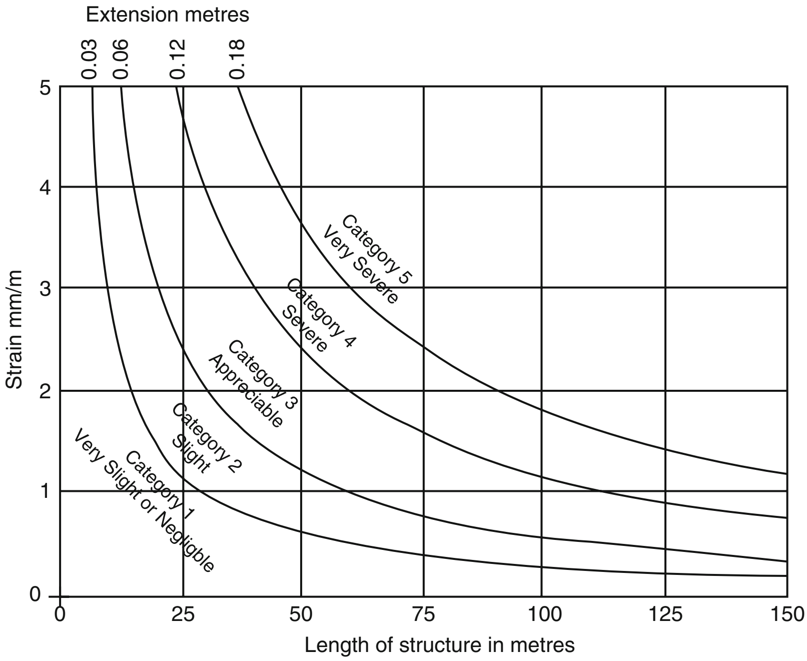
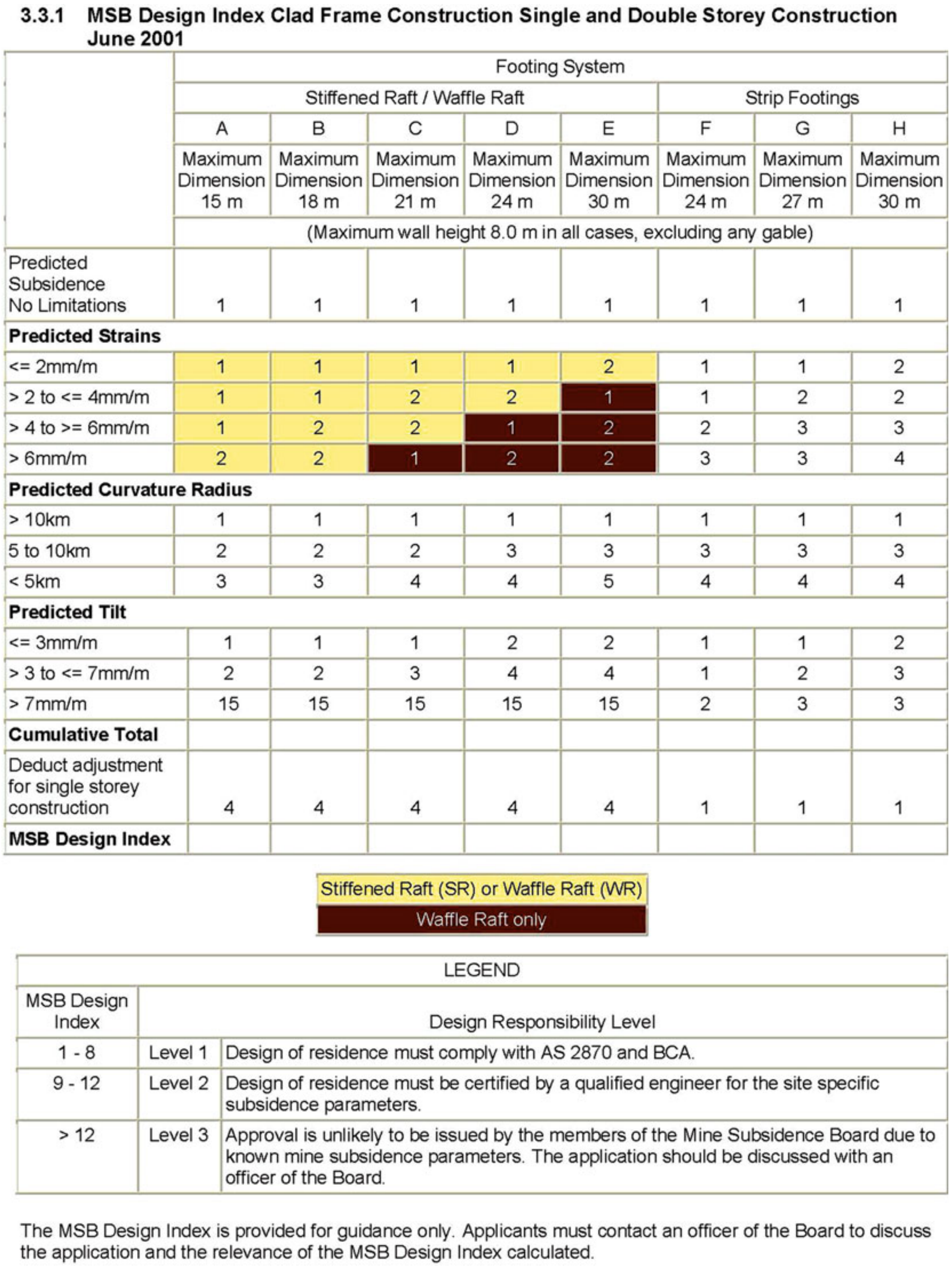


**Table A12.2** Graduated design guidelines of NSW Mine Subsidence Board as at 2001



### 3.3.1 MSB Design Index Clad Frame Construction Single and Double Storey Construction June 2001

| | Footing System | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Stiffened Raft / Waffle Raft | | | | | Strip Footings | | |
| | A | B | C | D | E | F | G | H |
| | Maximum Dimension 15 m | Maximum Dimension 18 m | Maximum Dimension 21 m | Maximum Dimension 24 m | Maximum Dimension 30 m | Maximum Dimension 24 m | Maximum Dimension 27 m | Maximum Dimension 30 m |
| | (Maximum wall height 8.0 m in all cases, excluding any gable) | | | | | | | |
| Predicted Subsidence No Limitations | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 |
| **Predicted Strains** | | | | | | | | |
| <= 2mm/m | 1 | 1 | 1 | 1 | 2 | 1 | 1 | 2 |
| > 2 to <= 4mm/m | 1 | 1 | 2 | 2 | 1 | 1 | 2 | 2 |
| > 4 to >= 6mm/m | 1 | 2 | 2 | 1 | 2 | 2 | 3 | 3 |
| > 6mm/m | 2 | 2 | 1 | 2 | 2 | 3 | 3 | 4 |
| **Predicted Curvature Radius** | | | | | | | | |
| > 10km | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 |
| 5 to 10km | 2 | 2 | 2 | 3 | 3 | 3 | 3 | 3 |
| < 5km | 3 | 3 | 4 | 4 | 5 | 4 | 4 | 4 |
| **Predicted Tilt** | | | | | | | | |
| <= 3mm/m | 1 | 1 | 1 | 2 | 2 | 1 | 1 | 2 |
| > 3 to <= 7mm/m | 2 | 2 | 3 | 4 | 4 | 1 | 2 | 3 |
| > 7mm/m | 15 | 15 | 15 | 15 | 15 | 2 | 3 | 3 |
| **Cumulative Total** | | | | | | | | |
| Deduct adjustment for single storey construction | 4 | 4 | 4 | 4 | 4 | 1 | 1 | 1 |
| **MSB Design Index** | | | | | | | | |

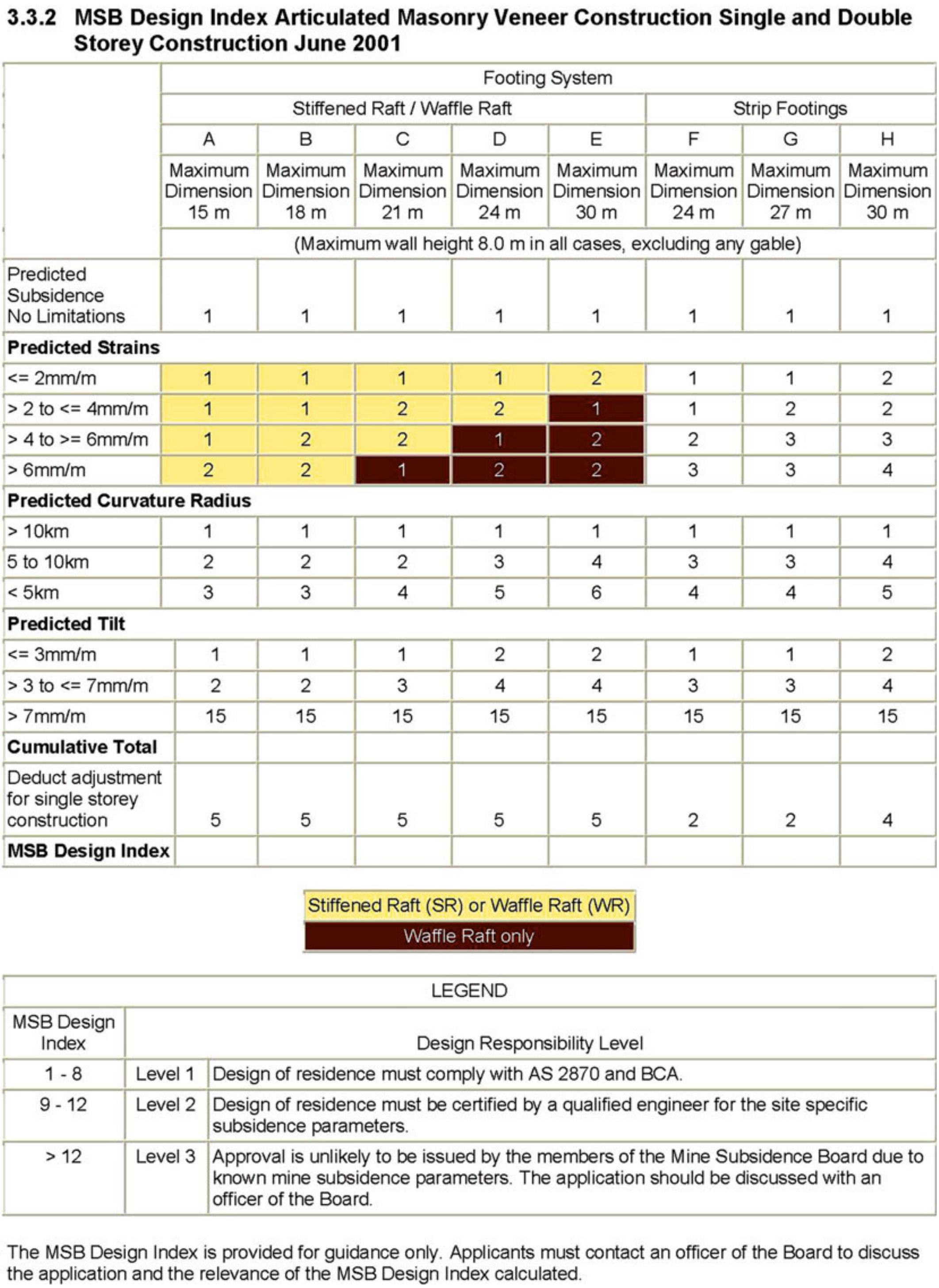
Stiffened Raft (SR) or Waffle Raft (WR)
Waffle Raft only

| LEGEND | | |
|---|---|---|
| MSB Design Index | Design Responsibility Level | |
| 1 - 8 | Level 1 | Design of residence must comply with AS 2870 and BCA. |
| 9 - 12 | Level 2 | Design of residence must be certified by a qualified engineer for the site specific subsidence parameters. |
| > 12 | Level 3 | Approval is unlikely to be issued by the members of the Mine Subsidence Board due to known mine subsidence parameters. The application should be discussed with an officer of the Board. |

The MSB Design Index is provided for guidance only. Applicants must contact an officer of the Board to discuss the application and the relevance of the MSB Design Index calculated.

(continued)

**Table A12.2** (continued)



### 3.3.2 MSB Design Index Articulated Masonry Veneer Construction Single and Double Storey Construction June 2001

| | Footing System | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Stiffened Raft / Waffle Raft | | | | | Strip Footings | | |
| | A | B | C | D | E | F | G | H |
| | Maximum Dimension 15 m | Maximum Dimension 18 m | Maximum Dimension 21 m | Maximum Dimension 24 m | Maximum Dimension 30 m | Maximum Dimension 24 m | Maximum Dimension 27 m | Maximum Dimension 30 m |
| | (Maximum wall height 8.0 m in all cases, excluding any gable) | | | | | | | |
| Predicted Subsidence No Limitations | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 |
| **Predicted Strains** | | | | | | | | |
| <= 2mm/m | 1 | 1 | 1 | 1 | 2 | 1 | 1 | 2 |
| > 2 to <= 4mm/m | 1 | 1 | 2 | 2 | 1 | 1 | 2 | 2 |
| > 4 to >= 6mm/m | 1 | 2 | 2 | 1 | 2 | 2 | 3 | 3 |
| > 6mm/m | 2 | 2 | 1 | 2 | 2 | 3 | 3 | 4 |
| **Predicted Curvature Radius** | | | | | | | | |
| > 10km | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 |
| 5 to 10km | 2 | 2 | 2 | 3 | 4 | 3 | 3 | 4 |
| < 5km | 3 | 3 | 4 | 5 | 6 | 4 | 4 | 5 |
| **Predicted Tilt** | | | | | | | | |
| <= 3mm/m | 1 | 1 | 1 | 2 | 2 | 1 | 1 | 2 |
| > 3 to <= 7mm/m | 2 | 2 | 3 | 4 | 4 | 3 | 3 | 4 |
| > 7mm/m | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 |
| **Cumulative Total** | | | | | | | | |
| Deduct adjustment for single storey construction | 5 | 5 | 5 | 5 | 5 | 2 | 2 | 4 |
| **MSB Design Index** | | | | | | | | |

Stiffened Raft (SR) or Waffle Raft (WR)

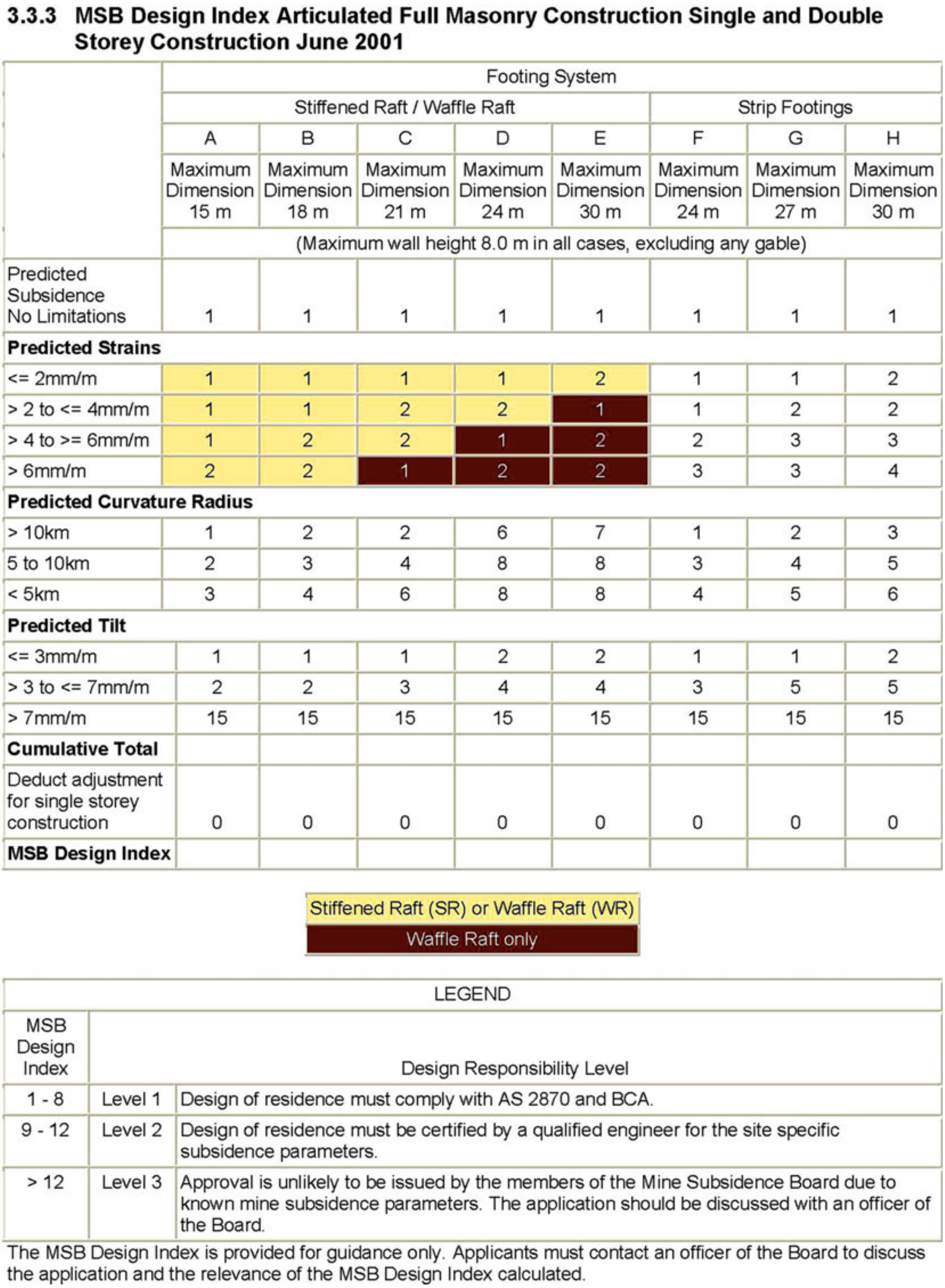
Waffle Raft only

| LEGEND | | |
|---|---|---|
| MSB Design Index | Design Responsibility Level | |
| 1 - 8 | Level 1 | Design of residence must comply with AS 2870 and BCA. |
| 9 - 12 | Level 2 | Design of residence must be certified by a qualified engineer for the site specific subsidence parameters. |
| > 12 | Level 3 | Approval is unlikely to be issued by the members of the Mine Subsidence Board due to known mine subsidence parameters. The application should be discussed with an officer of the Board. |

The MSB Design Index is provided for guidance only. Applicants must contact an officer of the Board to discuss the application and the relevance of the MSB Design Index calculated.

(continued)

**Table A12.2** (continued)



### 3.3.3 MSB Design Index Articulated Full Masonry Construction Single and Double Storey Construction June 2001

| | Footing System | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Stiffened Raft / Waffle Raft | | | | | Strip Footings | | |
| | A | B | C | D | E | F | G | H |
| | Maximum Dimension 15 m | Maximum Dimension 18 m | Maximum Dimension 21 m | Maximum Dimension 24 m | Maximum Dimension 30 m | Maximum Dimension 24 m | Maximum Dimension 27 m | Maximum Dimension 30 m |
| | (Maximum wall height 8.0 m in all cases, excluding any gable) | | | | | | | |
| Predicted Subsidence No Limitations | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 |
| **Predicted Strains** | | | | | | | | |
| <= 2mm/m | 1 | 1 | 1 | 1 | 2 | 1 | 1 | 2 |
| > 2 to <= 4mm/m | 1 | 1 | 2 | 2 | 1 | 1 | 2 | 2 |
| > 4 to >= 6mm/m | 1 | 2 | 2 | 1 | 2 | 2 | 3 | 3 |
| > 6mm/m | 2 | 2 | 1 | 2 | 2 | 3 | 3 | 4 |
| **Predicted Curvature Radius** | | | | | | | | |
| > 10km | 1 | 2 | 2 | 6 | 7 | 1 | 2 | 3 |
| 5 to 10km | 2 | 3 | 4 | 8 | 8 | 3 | 4 | 5 |
| < 5km | 3 | 4 | 6 | 8 | 8 | 4 | 5 | 6 |
| **Predicted Tilt** | | | | | | | | |
| <= 3mm/m | 1 | 1 | 1 | 2 | 2 | 1 | 1 | 2 |
| > 3 to <= 7mm/m | 2 | 2 | 3 | 4 | 4 | 3 | 5 | 5 |
| > 7mm/m | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 |
| **Cumulative Total** | | | | | | | | |
| Deduct adjustment for single storey construction | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| **MSB Design Index** | | | | | | | | |

Stiffened Raft (SR) or Waffle Raft (WR)

Waffle Raft only

| LEGEND | | |
|---|---|---|
| MSB Design Index | Design Responsibility Level | |
| 1 - 8 | Level 1 | Design of residence must comply with AS 2870 and BCA. |
| 9 - 12 | Level 2 | Design of residence must be certified by a qualified engineer for the site specific subsidence parameters. |
| > 12 | Level 3 | Approval is unlikely to be issued by the members of the Mine Subsidence Board due to known mine subsidence parameters. The application should be discussed with an officer of the Board. |

The MSB Design Index is provided for guidance only. Applicants must contact an officer of the Board to discuss the application and the relevance of the MSB Design Index calculated.

**Table A12.3** Revised classification of subsidence impacts to houses based on extent of repairs (Waddington 2009)

| Repair category | Extent of repairs |
|---|---|
| Nil | No repairs required |
| R0 adjustment | One or more of the following, where the damage does not require the removal or replacement of any external or internal claddings or linings: |
| | Door or window jams or swings, or |
| | Movement of cornices, or |
| | Movement at external or internal expansion joints |
| R1 very minor repair | One or more of the following, where the damage can be repaired by filling, patching or painting without the removal or replacement of any external or internal brickwork, claddings or linings:- |
| | Cracks in brick mortar only, or isolated cracked, broken, or loose bricks in the external façade, or |
| | Cracks or movement $< 5$ mm in width in any external or internal wall claddings, linings, or finish, or |
| | Isolated cracked, loose, or drummy floor or wall tiles, or |
| | Minor repairs to any services or gutters |
| R2 minor repair | One or more of the following, where the damage affects a small proportion of external or internal claddings or linings, but does not affect the integrity of external brickwork or structural elements:- |
| | Continuous cracking in bricks $<5$ mm in width in one or more locations in the total external façade, or |
| | Slippage along the damp proof course of 2–5 mm anywhere in the total external façade, or |
| | Cracks or movement $\geq 5$ mm in width in any external or internal wall claddings, linings, finish, or |
| | Several cracked, loose or drummy floor or wall tiles, or |
| | Replacement of any services |
| R3 substantial repair | One or more of the following, where the damage requires the removal or replacement of a large proportion of external brickwork, or affects the stability of isolated structural elements:- |
| | Continuous cracking in bricks of 5–15 mm in width in one or more locations in the total external façade, or |
| | Slippage along the damp proof course of 5–15 mm anywhere in the total external façade, or |
| | Loss of bearing to isolated walls, piers, columns, or other load-bearing elements, or |
| | Loss of stability of isolated structural elements |
| R4 extensive repair | One or more of the following, where the damage requires the removal or replacement of a large proportion of external brickwork, or the replacement or repair of several structural elements:- |
| | Continuous cracking in bricks $> 15$ mm in width in one or more locations in the total external façade, or |
| | Slippage along the damp proof course of 15 mm or greater anywhere in the total external façade, or |
| | Relevelling of building, or |
| | Loss of stability of several structural elements |
| R5 Re-build | Extensive damage to house where the MSB and the owner have agreed to rebuild as the cost of repair is greater than the cost of replacement |

**Table A12.4** Probabilities of subsidence impacts based on curvature and type of construction (Waddington 2009)

| | Repair category | | | |
|---|---|---|---|---|
| Radius of curvature (km) | No repair or R0 | R1 or R2 | R3 or R4 | R5 |
| **Brick or brick-veneer houses with Slab on Ground** | | | | |
| >50 | 90 ~ 95 % | 3 ~ 10 % | 1 % | <0.1 % |
| 15–50 | 80 ~ 85 % | 12 ~ 17 % | 2 ~ 5 % | <0.5 % |
| 5–15 | 70 ~ 75 % | 17 ~ 22 % | 5 ~ 8 % | <0.5 % |
| **Brick or brick-veneer houses with Strip Footing** | | | | |
| >50 | 90 ~ 95 % | 3 ~ 10 % | 1 % | <0.1 % |
| 15–50 | 80 ~ 85 % | 7 ~ 12 % | 2 ~ 7 % | <0.5 % |
| 5–15 | 70 ~ 75 % | 15 ~ 20 % | 7 ~ 12 % | <0.5 % |
| **Timber-framed houses with flexible external linings of any foundation type** | | | | |
| >50 | 90 ~ 95 % | 3 ~ 10 % | 1 % | <0.1 % |
| 15–50 | 85 ~ 90 % | 7 ~ 13 % | 1 ~ 3 % | <0.5 % |
| 5–15 | 80 ~ 85 % | 10 ~ 15 % | 3 ~ 5 % | <0.5 % |

## References

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Waddington, A. A. (2009). *The prediction of mining induced movements in building structures and the development of improved methods of subsidence impact assessment*. ACARP end of project report no. C12015. Brisbane: Australian Coal Association Research Program (ACARP).

# Appendix 13: Examples of Risk Management Based Statutory Requirements Relevant to Developing Ground Control Management Plans

## NSW Coal Mine Health and Safety Regulation, 2006

### 32 Contents of Major Hazard Management Plan: Strata Failure Management Plan

For the purposes of section 36 of the Act, a major hazard management plan in relation to a major hazard comprising hazards arising from strata failure must make provision for the following matters:

(a) the estimation of the geological conditions likely to be encountered in roadway development,
(b) the assessment of the stability of roadways to be developed in those geological conditions,
(c) the recording of geological conditions that may affect roadway stability,
(d) the development of support measures that will provide roadway stability in those geological conditions,
(e) calculations (including maximum roadway width and the minimum dimensions of coal pillars) to determine the probability of instability to be assigned to any coal pillar, consistent with the pillar's role or roles over its life,
(f) the preparation and distribution of support plans that clearly describe the following:
    (i) the type of support,
    (ii) the dimensions of the support,
    (iii) the locations where there are varying types of supports in use,
    (iv) the distance between supports,
    (v) the maximum distance roadways can be advanced before support is installed,
    (vi) the means of roadway support required to be installed in a manner such that they may be readily understood by those required to install the roadway support,
(g) other information necessary to enable an employee to install support according to the requirements of the management plan,
(h) safe, effective and systematic work methods for the installation, and subsequent removal when required, of the roadway support (including support in connection with the carrying out of roof brushing operations or the recovery of plant),
(i) the availability of adequate plant and resources to effectively install or remove the roadway support,
(j) the monitoring of the stability of roadways after development and support installation,
(k) the training of employees in support design principles, support plan interpretation, placement and removal of support, understanding the need for and the

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importance of the various support systems and recognition of indicators of change that may affect roadway stability,

- (l) the recording of strata failures that have the potential to cause serious injury (such as a notifiable incident under clause 55) to people,
- (m) a description of the following features and any special provision made for them:
  - (i) any multi-seam workings,
  - (ii) any mining that has the potential to cause windblast or rapid stress change,
  - (iii) any mining at depths of less than 50 m,
  - (iv) any coal pillars with a pillar width to pillar height ratio of 4:1 or less,
- (n) a prohibition on people entering an underground place at the coal operation that is not supported in accordance with the management plan, unless the person does so for the purpose of erecting support, in which case temporary support must be used,
- (o) a prohibition on mining in any place at the coal operation unless there is sufficient support for the place in accordance with the requirements of the management plan,
- (p) a statement that nothing in the management plan is to be read as preventing the installation of more strata support or support installation at more frequent intervals than is required by the management plan.

## QLD Coal Mine Safety and Health Regulation, 2001

### 317 Risk Assessment

1. The underground mine manager must ensure a risk assessment is carried out under this section to decide a safe method of extraction for second workings at the mine before the second workings start.
2. The risk assessment must have regard to at least the following matters—
   - (a) any surface features, artificial structures and water reserves that may create a hazard if disturbed by the workings;
   - (b) any other workings located in close proximity above, below or adjacent to the proposed second workings, whether in the same or an adjacent mine;
   - (c) the known geology affecting the intended workings;
   - (d) the anticipated gas make;
   - (e) the pillar stability;
   - (f) the proposed method and sequence of coal extraction;
   - (g) the proposed methods for the following—
     - (i) strata control and support;
     - (ii) ventilation;
     - (iii) controlling spontaneous combustion;
   - (h) support methods necessary to control the edges of each goaf area in active workings;
   - (i) the suitability of the plant, and its controls, used for the workings.

### 318 Standard Operating Procedure

1. An underground mine must have a standard operating procedure for carrying out second workings.
2. The procedure must be based on the results of the risk assessment mentioned in section 317.
3. The mine must have a separate procedure for each panel in the mine.
4. However, if the hazards in each panel in a group of panels are the same, the mine may have a standard operating procedure for the group.
5. The procedure must include provision for establishing—
   - (a) methods for the following—
     - (i) coal extraction;
     - (ii) strata control and support;
     - (iii) ventilation;

    - (iv) controlling spontaneous combustion;
    - (v) monitoring and recording extraction progress; and
  - (b) the coal extraction sequence.

## 319 Changing Standard Operating Procedure

1. This section applies to an underground mine if -
   - (a) the conditions or hazards in a panel, or group of panels, in the mine changes significantly while coal is being extracted in the panel or group in second workings; or
   - (b) it is proposed to significantly change a method for the workings established under section 318(5)(a).
2. If subsection (1)(a) applies—
   - (a) the underground mine manager must ensure a risk assessment for the workings is carried out as soon as practicable after the change happens; and
   - (b) the standard operating procedure for carrying out the workings in the panel, or group of panels, must be reviewed and, based on the risk assessment, amended, if necessary.
3. If subsection (1)(b) applies, before the change is implemented—
   - (a) the underground mine manager must ensure a risk assessment is carried out for the proposed change; and
   - (b) the standard operating procedure for carrying out the workings must be amended, if necessary, based on the risk assessment.

# Appendix 14: Sources of Information Relevant to Managing Risk in Ground Engineering

| ORGANISATIONS | |
|---|---|
| International Labour Organisation | www.ilo.org |
| Safe Work Australia | www.safeworkaustralia.gov.au |
| Minerals Council of Australia | www.minerals.org.au |
| NSW Minerals Council | www.nswmc.com.au |
| Queensland Mining Council | www.qmc.com.au |
| Tasmanian Minerals Council | www.tasminerals.com.au |
| W.A. Chamber of Mines and Energy | www.mineralswa.asn.au |
| Mine Managers Association of Australia | www.minemanagers.com.au |
| Minerals Industry Safety and Health Centre (MISHC) | www.mishc.uq.edu.au |
| Minerals Industry Risk Management Gateway | www.mirmgate.com |
| Australian Coal Association Research Program | www.acarp.com.au |
| Cooperative Research Centre Mining | http://www.crcmining.com.au/ |
| Ministry of Labour Occupational Health and Safety - Canada | www.gov.on.ca/lab/ohs |
| Mine Safety and Health Administration of the US Department of Labour | www.msha.gov |
| National Institute for Occupational Safety and Health (NIOSH) - USA | www.cdc.gov/niosh/ |
| Health and Safety Executive - UK | www.hse.gov.uk/ |
| Mine Health and Safety Council, South Africa | http://www.mhsc.org.za/ |
| Safety in Mining Research Dept. Science and Technology – Sth Africa | http://www.dst.gov.za/s-t-landscape/S-T%20Funding%20Agencies/SIMRAC |

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| STANDARDS | |
|---|---|
| **International Labour Organisation** | www.ilo.org |
| ILO C176 | Safety and Health in Mines Convention, 1995 (No. 176). |
| **US Standards** | |
| ASTM F432-10 | Standard Specification for Roof and Rock Bolts and Accessories |
| **Australian/NZ/International Standards** | www.standards.com.au |
| AS/NZS/ISO 9001: 2008 | Quality Management Systems - Requirements |
| AS/NZS/ISO 31000: 2009 | Risk Management - Principles and Guidelines |
| **Australian Standards** | www.standards.com.au |
| AS 1470-1986 | Health and Safety at Work – Principles and Practices |
| AS 1614-1985 | The Design and Use of Reflectorized Safety Signs for Mines and Tunnels |
| AS/NZS 3905.12: 1999 | Quality System Guidelines. Part 12: Guide to AS/NZS 9001:1994 for Architectural and Engineering Design Practice |
| AS/NZS 3931-1998 | Risk Analysis of Technological Systems – Application Guide. |
| AS 4024-1996 | Safeguarding of Machinery |
| AS 4368-1996 | Mine Plans – Preparation and Symbols |
| AS/NZS 4801:2001 | Occupational Health and Safety Management Systems |
| AS/NZS 4804:2001 | Occupational Health and Safety Management Systems – General Guidelines on Principles, Systems and Supporting Techniques |

| **GUIDELINES** | |
|---|---|
| **Australian Government** | http://www.safeworkaustralia.gov.au/sites/SWA |
| | Code of Practice for Strata Control in Underground Coal Mines |
| | Code of Practice for Inundation and Inrush Hazard Management |
| **NSW Government** | http://www.resources.nsw.gov.au/ |
| MDG 5 | Code for Portable Pneumatically Powered Rotary Roof Bolters for Use in Coal Mines |
| MDG 6 | Code for Breaker Line Supports for Use in Coal Mines |
| MDG 11 | Design Guidelines for the Use of Aluminium in Underground Coal Mines |
| MDG 1003 | Wind Blast Code of Practice |
| MDG 1004 | Outburst Mining Guideline |
| MDG 1005 | Manual on Pillar Extraction |
| MDG 1006 | Spontaneous Combustion Management Code |
| MDG 1007 | Explosion at Endeavour Colliery |
| MDG 1010 | Risk Management Handbook for the Mining Industry |
| MDG 1013 | Systems Safety Technique |
| MDG 1014 | Guide to Reviewing a Risk Assessment of Mine Equipment and Operations |
| MDG 1017 | Roof Support Guidelines for Massive Strata Conditions |
| MDG 1022 | Guidelines for Determining Withdrawal Conditions from Underground Coal Mines |
| MDG 1030 | Guideline for Raise Boring Operations |
| MDG 3001 | Applicant's Guide to Obtaining an Approval from the Chief Inspector of Coal Mines |
| MDG 3002 | Systems Safety Accident Investigation Series |
| MDG 3003 | Summaries of Reportable Accidents and Dangerous Occurrences: 1991 onwards. |
| MDG 3004 | Review of Reportable Frictional Ignitions of Methane in New South Wales Underground Coal Mines |
| MDG 3012 | Safety Alerts and Significant Incident Reports to 1996 |
| - | Safety Alerts – Post 1996 |
| MDG 5002 | Mine Safety Review Guidelines for the Use of Remote Controlled Mining Equipment |
| MDG 5003 | Guidelines for Contractor Occupational Health and Safety Management for New South Wales Coal Mines |
| EDG 1 | Guidelines for Borehole Sealing Requirements on Land – Coal Exploration |
| EDG 2 | Guidelines for Borehole Sealing Requirements on the Beds of Waterbodies – Coal Exploration |
| - | Minerals Industry Safety Handbook |
| **Western Australian Government** | www.doir.wa.gov.au |
| ZMT723RK | Underground Barring Down and Scaling Guideline |
| **South African Government** | www.dmr.gov.za |
| DME 16/3/2/1-A3 | Guideline for the Compilation of a Mandatory Code of Practice to Combat Rock Falls and Rockburst Accidents in Tabular Metalliferous Mines |

| **LEGISLATION AND REGULATION** | |
|---|---|
| **NORTH AMERICA** | |
| Mine Safety and Health Administration of the US Department of Labour | www.msha.gov |
| Ministry of Labour Occupational Health and Safety - Canada | www.gov.on.ca/lab/ohs |
| **UK** | |
| Health and Safety Executive - UK | www.hse.gov.uk/ |
| **AUSTRALIA** | |
| Australian Legal Information Institute | www.austlii.edu.au |
| Queensland Legislation | www.legislation.qld.gov.au/Legislation.htm |
| NSW Legislation | www.legislation.nsw.gov.au/ |
| Qld Department of Natural Resources and Mines | www.nrm.qld.gov.au/mines |
| Mining Warden’s Court of Queensland | www.warden.qld.gov.au |
| WorkCover - Queensland | www.workcover.qld.gov.au |
| NSW Minerals and Petroleum | http://www.resources.nsw.gov.au/ |
| WorkCover - NSW | www.workcover.nsw.gov.au |
| Conference of Chief Inspectors of Mines | www.agso.gov.au/ccim |

# Appendix 15: Guidelines for Developing a Mine Safety Management System and a Principal Hazard Management Plan

## Guidance Material

The following summary is based on guidelines for preparing a Mine Safety Management System (MSMS) and a Hazard Management Plan (HMP) produced by the Queensland Department of Mines & Energy (Qld Dept. Mines & Energy 1996).

## Health and Safety Management System

### Purpose of a MSMS

A Mine Safety Management System (MSMS) is intended to formalize the process by which a mine addresses its principal hazards and other related matters in order to provide a safe work place. A MSMS is expected to:

1. Set out the means by which hazards are identified and risks are assessed.
2. Identify effective risk control measures which are independent of changes in site personnel.
3. Ensure consistency in the way hazards are controlled.
4. Set performance standards.
5. Provide for the monitoring of performance standards.
6. Set safety objectives for the mine.
7. Detail the system for achieving safety objectives.
8. Provide for detecting changes in conditions.
9. Detail procedures for the conduct of regular reviews of the operation and adequacy of the system.

A MSMS is not intended to be an additional layer of regulation. Nor does such a system remove any obligation to fulfil statutory requirements. Rather, its purpose is to provide the framework within which a mine site can manage risk, consistent with its statutory obligations, but with the actual system content and ownership residing with the mine site. As such, it is intended to be proactive and broader ranging approach than that required by prescriptive legislation.

## Structure of a MSMS

A MSMS should:

1. Provide a systematic definition of all the actions necessary to allow mining operations to be carried out safely;
2. Include, but not be limited to, organizational structures, planning, activities, responsibilities, practices, risk identification, audits and reviews;

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3. Be based on appropriate codes, standards, rules, regulations and guidelines.

Therefore, a MSMS could be expected to include, but not be limited to, the following elements:

## Introduction

The Introduction should state the overall objective of the MSMS, establish the framework and ownership of the document and provide guidance as to its use. It may include corporate safety targets and objectives and details of the structure of the organisation, including roles and responsibilities of members of the organisation.

## Scope

The Scope should list all the Hazard Management Plans (HMPs) which apply at the mine. In general, the hazards should all be identified though a process of risk assessment. In practice, however, some hazards are mandated by legislation because they present core risks to mining operations. Examples of mandated hazards for which underground coal mines must prepare a HMP include:

- Mine Atmosphere, incorporating:
  - Ventilation Management
  - Gas Management
- Strata Management
- Spontaneous Combustion Management
- Fire Control
- Emergency Response

## Mine Characteristics

This section should outline the characteristics of the mine which are relevant to the various HMPs, taking into account both current and future mine design. The types of matters which should be briefly described include:

- Access to the mine
- Seams mined
- Depth of mining
- Seam characteristics
- Mining methods
- Ventilation
- Shift working systems
- Employment

## Identification of Principal Hazards

The methods by which the principal hazards were identified, who was involved in the process, positions title and qualifications of these persons, the scope of the hazard analysis and risk assessment, and the identified principal hazards should be documented in this section of the MSMS. The intent is to establish that the personnel involved in the hazard identification were appropriately qualified, experienced and competent and that the process was adequately resourced and undertaken in accordance with an established risk assessment standard.

## Organisational Responsibilities and Resources

The purpose of this section is to demonstrate that the organization has allocated sufficient resources and assigned appropriate responsibilities to fulfill the requirements of the MSMS. Each HMP should have a document owner who is responsible for administration and co-ordination of the HMP.

## Management Review

This section should detail Senior Management's responsibility and commitment to review the

MSMS at regular intervals in order to ensure its continuing suitability and effectiveness, to identify new hazards where appropriate, and to implement improvements and corrective actions.

## Developing a Hazard Management Plan

A Hazard Management Plan should be developed in support of the Mine Safety Management System to address each principal hazard encountered or likely to be encountered at a mine. It could be expected to include, but not be limited to, the following elements:

### Introduction

States the objective and scope of the HMP with respect to the specific hazard being addressed. Objectives usually encompass the formalization of the systems, standards, procedures and methods in use or to be introduced to ensure effective management and control of the hazard.

### Identified Hazard

Outlines the method by which the hazard was identified and assessed.

### Control Procedures

Identifies the control procedures to be followed and the persons responsible for implementing each control. The details which comprise each control procedure may be contained in separate supporting documents. A control procedure may find application to more than one hazard.

### Roles and Responsibilities

This section should outline the roles, responsibilities and competencies of all persons having accountability under the HMP. This includes internal personnel and external providers. The assignment of responsibilities to a person should take into account any statutory obligations of this person.

### Resources Required

Identifies the resources that the organisation has to put in place in order to meet the requirements of the HMP.

### Trigger Action Response Plans (TARPs)

These plans outline trigger points, the actions to be taken for each trigger point and the persons responsible for implementing these actions. Trigger points can include observations, measurements or events.

### Communications

Details the establishment and maintenance of procedures for:

(a) Internal communication between the various levels and functions of the mine.
(b) Receiving, documenting and responding to relevant communications to the hazard being addressed.

### Training

Identifies the training required to address a specific hazard, the persons who are to receive this training, and the establishment and maintenance of procedures to make employees at each relevant function and level aware of:

(a) The importance of conformance with procedures and with the requirements of the HMP;
(b) The significance safety impacts, actual or potential, of their work activities and the

safety benefits of improved personal performance;
(c) The roles and responsibilities in achieving conformance with procedures and with the requirements of the HMP, including emergency preparedness and response requirements;
(d) The potential consequences of departure from the HMP.

## Corrective Action

Details the establishment and maintenance of procedures for:

(a) defining responsibility and authority for handling and investigating non-conformance;
(b) taking action to mitigate any impacts caused;
(c) for initiating and completing corrective and preventative actions;
(d) implementing any changes in documented procedures as a result of corrective and preventative actions;
(e) recording any changes in documented procedures.

## Review

Details:

(a) The intervals at which the HMP is to be reviewed to ensure its continuing suitability, adequacy and effectiveness.
(b) Procedures for reviewing significant issues which arise and their response status.

## Audit

Details the establishment and maintenance of programmes and procedures for periodically auditing and reviewing the HMP in order to determine whether or not the HMP:

(a) Conforms to planned arrangements for safety management;
(b) Has been properly implemented and maintained.

This element should include details on how management is to be informed of the results of audits and reviews.

## Document Control

Details the establishment and maintenance of procedures which:

(a) Assign responsibilities and define processes for the creation and modification of documents relevant to the HMP;
(b) Ensure that all documents relevant to the HMP are:
    (i) Legible;
    (ii) Dated, including dates of revision;
    (iii) Readily identifiable;
    (iv) Maintained in an orderly manner;
    (v) Maintained for any specified period;
    (vi) Readily locatable and accessible;
    (vii) Periodically reviewed, revised as necessary and approved for adequacy by authorised personnel;
    (viii) Available, as a current version, at all locations where operations essential to the effective functioning of the plan are performed;
(c) Ensure that obsolete documents related to the HMP are:
    (i) All promptly removed from all points of issue and points of use or otherwise assured against unintended use;
    (ii) Suitably identified as being obsolete if they are retained for purposes such as legal proceedings or knowledge preservation.

## Records

Details the establishment and maintenance of procedures for the identification, maintenance and deposition of health and safety related records, including training records and the results of audits and reviews. Reference should be made to provisions for ensuring that safety records:

(a) Are legible, identifiable and traceable to the activity involved;
(b) Stored and maintained in such a way that they are readily retrievable and protected against damage, deterioration or loss;
(c) Have an established retention time which is recorded.

## References

Qld Department of Mines & Energy. (1996). *Approved standard for mine safety management plan*. QMD 967386/A. Brisbane: Department of Mines and Energy, Qld State Government.

# Appendix 16: An Example of a Trigger Action Response Plan (Ground Management on a Longwall Face)

**TRIGGER ACTION RESPONSE PLAN – LONGWALL FACE**

| | | Level 1 – Condition Green | Level 2 – Condition Yellow | Level 3 – Condition Orange | Level 4 – Condition Red |
|---|---|---|---|---|---|
| **TRIGGER** | **Roof Conditions** | **Geology**<br>1. Minor geological structure to 0.5 m | **Geology**<br>1. Faults converging within 5 to 10 chocks of each other<br>2. Faults within 5 chocks of gateroads<br>3. Major sandstone lens within 10m of coal seam | **Geology**<br>1. Faults converging within 5 chocks of each other<br>2. Major sandstone unit within 5m of coal seam | -- |
| | | **Roof Coal Thickness**<br>1. Greater than 1.0m | **Roof Coal Thickness**<br>1. Less than 1.0m | **Roof Coal Thickness**<br>1. None. Stone visible over up to 10 consecutive chocks but stable | **Roof Coal Thickness**<br>1. None. Stone visible over more than 10 consecutive chocks and continues to dribble |
| | | **Visual**<br>1. Normal conditions. Pick marks visible and remain in cut roof or visible parting at desired horizon<br>2. Break line at rear edge of chocks | **Visual**<br>1. Roof deteriorating. Fretting, loss of visible pick marks. Loss of natural parting above desired cut horizon<br>2. Break line over canopy, forward of legs | **Visual**<br>1. Roof guttering or roof falling to stone bands <1m above cut roof<br>2. Break line between canopy tips and face | **Visual**<br>1. Roof fall greater than 1m above cut horizon and hading at least 1 web ahead of face. Large quantities of material continue to rill in<br>2. Break line ahead of face. Face being scoured out by falling material |
| | | **Tip-to-Face**<br>1. Less than 0.75m after chocks advanced | **Tip-to-Face**<br>1. Between 0.75 and 1.2m after chocks advanced | **Tip-to-Face**<br>1. Between 1.2m and 1.5m after chocks advanced | **Tip-to-Face**<br>1. Greater than 1.5m after chocks advanced |
| | | **Chock Set Pressure**<br>1. 350 – 400 Bar | **Chock Set Pressure**<br>1. Less than 350 Bar | **Chock Set Pressure**<br>1. Less than 200 Bar over 5 or more consecutive chocks | **Chock Set Pressure** |
| | | **Chock Convergence**<br>1. <50mm/hour<br>2. No flow or only a few drips from yield valves | **Chock Convergence**<br>1. Greater than 50mm/hour but less than 100mm/hour<br>2. Some minor fluid flow from yield valves over a length of 15 chocks | **Chock Convergence**<br>1. Greater than 100mm/hour but less than 200mm/hour<br>2. Continuous fluid flow from yield valves over a length of 15 chocks | **Chock Convergence**<br>1. Greater than 200mm/hour<br>2. Shearer barely passes through under canopies (nearly iron bound)<br>3. Continuous fluid flow from yield valves over a length of >15 chocks |
| | **Face Conditions** | **Visual**<br>1. Some face slabbing ahead of leading drum<br>2. Face spall less than 0.5m | **Visual**<br>1. Minimal cutting required with spall occurring greater than 10 chocks ahead of leading drum<br>2. Spall 0.5 to 1.0m | **Visual**<br>1. Face slabbing heavily with heavy spall well in advance of leading drum<br>2. Face spall 1.1 to 1.5m | **Visual**<br>1. Face slabbing heavily with heavy spall well in advance of leading drum and behind trailing drum<br>2. Face spall greater than 1.5m |
| **RESPONSE** | **Mode of Operation** | 1. Uni-Di with shearer initiation | 1. Uni-Di with auto adjacent controls<br>2. Cancel shearer initiated chock advance and push primes in affected area<br>3. Restrict shearer speed to match chock operators through affected area | 1. Uni-Di manual control<br>2. Bi-Di sequence optional in localised areas<br>3. Cancel shearer initiated chock advance and push primes in affected area<br>4. Monitor AFC flow to BSL to prevent blockages<br>5. Restrict shearer speed to match chock operations through affected areas | 1. Keep shearer on tailgate side of affected area during chocking<br>2. Employ Uni-Di manual control if sufficient clearance<br>3. Cancel shearer initiated chock advance and push primes throughout affected area<br>4. Monitor AFC flow to BSL to prevent blockages<br>5. Restrict shearer speed to 50% max through affected area |
| | **Support Operation** | 1. Supports advanced 2 chocks behind leading drum.<br>2. Positive set in use | 1. Single support advance immediately behind the leading drum<br>2. Double chock where practical<br>3. Sprags set behind trailing drum<br>4. Positive set in use except where it affects canopy attitude. Guaranteed set used in these softer zones – attempt to maintain canopy attitude and maximise tip pressure. Ensure positive set is turned on to supports adjacent to yellow zone. Positive set to be turned on ASAP when conditions permit. | 1. Single support advance immediately behind leading drum<br>2. Double chock<br>3. Set sprags ASAP<br>4. Use positive set except where it affects canopy attitude. Chocks should otherwise be set with maximum pressure to maintain canopy attitude and to maximise tip pressure. Ensure positive set is turned on to supports outside of orange zone<br>6. Positive set to be turned on ASAP when conditions permit. | 1 Keep shearer on tailgate side of affected area during chocking<br>2. Use manual overrides through affected area<br>3. Double chock ASAP<br>4. Set sprags ASAP<br>5. Use positive set except where it affects canopy attitude. Chocks should otherwise be set with maximum pressure to maintain canopy attitude and to maximise tip pressure. Ensure positive set is turned on to supports outside of orange and red zones<br>6. Positive set to be turned on ASAP when conditions permit. |
| | **Face Alignment** | 1. Check face alignment each shift and correct as required | 1. Check face alignment each shift and correct as required | 1. Check face alignment each shift and correct as required under direction of Face Supervisor | 1. Check face alignment each shift and correct as required under direction of Face Supervisor after consultation with Longwall Superintendant |
| | **Horizon Control** | 1. Roof height and cutting horizon as per longwall hazard management plans | 1. Roof height and cutting horizon as per longwall hazard management plans | 1. Roof height and cutting horizon as per Longwall Superintendant's instructions. | 1. Roof height and cutting horizon as per Longwall Superintendant's instructions. |
| | **Creep Control** | 1. Continually check creep<br>2. Creep is <200mm offline subject to MG roadway alignment | 1. Continually check creep<br>2. Fly cut if creep is 200mm to 500mm from normal but only under Face Supervisor's instructions | 1. Continually check creep<br>2. Correct if creep is over 500mm from normal and getting worse but only under Longwall Superintendant instructions | 1. Continually check creep<br>2. Contact Longwall Superintendant before correcting creep in red affected areas |
| **AUTHORITY** | **Change Condition Up** | 1. The Face Supervisor has the authority to move to a higher trigger level | 1. The Face Supervisor has the authority to move to a higher trigger level | 1. The Face Supervisor has the authority to move to a red trigger level | -- |
| | **Change Condition Down** | -- | 1. The Face Supervisor has the authority to revert back to lower trigger level | 1. The Longwall Superintendant and Longwall Coordinator in consultation with the Geotechnical Engineer have the authority to revert back to a lower trigger level | 1. The Longwall Superintendant in consultation with the Geotechnical Engineer has the authority to revert back to a lower trigger level |

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**TRIGGER ACTION RESPONSE PLAN – LONGWALL FACE**

| | | Level 1 – Condition Green | Level 2 – Condition Yellow | Level 3 – Condition Orange | Level 4 – Condition Red |
|---|---|---|---|---|---|
| **RESPONSIBILITIES** | **Accountability** | When a person under a TARP is unavailable, that person's immediate supervisor is to fulfil the role | | | |
| | **Shearer Operator** | 1. Operate to set standards and Face Supervisor's Instructions<br>2. Observe for deteriorating conditions and report to Face Supervisor, chock operators and maingate operator<br>3. Maintain set shearer speed<br>4. Report chock defects to trades<br>5. Ensure positive set is on | 1. Cancel shearer initiated chock advance and push primes in affected area<br>2. Reduce shearer speed to match chock operators in affected area | 1. Cancel shearer initiated chock advance and push primes in affected area<br>2. Reduce shearer speed to match chock operators in affected area | 1. Keep shearer on tailgate side of affected area during chocking<br>2. Cancel shearer initiated chock advance and push primes in affected area<br>3. Reduce shearer speed to match chock operators in affected area |
| | **Chock Operators** | 1. Monitor chock automation | 1. Operate chocks in auto adjacent control with positive set isolated when it affects canopy attitude (maintain level canopy attitude)<br>2. Advance chocks two behind leading drum<br>3. Set sprags ASAP<br>3. Return to positive set ASAP | 1. Operate chocks in manual control with positive set isolated when it affects canopy attitude<br>2. Advance chocks immediately behind leading drum and set sprags ASAP<br>3. Double chock in affected area ASAP<br>3. Return to positive set ASAP | 1. Operate chocks in manual control with positive set isolated when it affects canopy attitude<br>2. Advance chocks immediately behind leading drum and set sprags ASAP<br>3. Double chock in affected area ASAP<br>3. Return to positive set ASAP |
| | **Maingate Operator** | 1. Monitor creep<br>2. Check system pump pressure, chock set pressures, positive set pressures, emulsion tank levels | -- | 1. Monitor AFC flow to BSL to prevent blockages | -- |
| | **Trades** | 1. Monitor chock conditions and repair or report as required<br>2. Check pump pressure<br>3. Identify and report defective legs and sprags in accordance with specified criteria | 1. Apply manual applications to overcome maintenance problems causing downtime in adverse conditions (i.e. bypass cooling water)<br>2. Ensure shear shafts available<br>3. Ensure TTT ram start operational | 1. Increase AFC tension when stone is present on conveyor | -- |
| | **Face Supervisor** | 1. Observe for changing conditions and action TARP<br>2. Record face conditions on shift report and identify all TARP triggered zones, yield or isolated positive set<br>3. Monitor leg pressures and emulsion tank levels<br>4. Ensure TARP standards are being followed | 1. Inform control room operator of condition of yellow affected areas<br>2. Continual monitoring of affected areas when mining within them<br>3. Communicate conditions to oncoming Face Supervisor | 1. Monitor and report chock and SIM clearance and convergence in yielding zone<br>2. Inform control room operator of condition orange affected areas<br>3. Continuously monitor affected area whilst cutting in that zone<br>4. Communicate conditions to on coming Face Supervisor | 1. Inform Conrol Room Operator of condition red areas<br>2. Ensure shearer is on tailgate side of cavity during chocking and that personnel are operating from a safe position<br>3. Ensure 5 competent operators are available for production<br>4. Continuously monitor affected area whilst cutting in that zone<br>5. Communicate conditions to on coming Face Supervisor |
| | **Control Room Operator** | 1. Monitor chock pressures and emulsion tank levels<br>2. Communicate irregularities in operating system to face supervisor | 1. Note changed conditions on shift report<br>2. Inform shift supervisor of changed conditions | 1. Note changed conditions on shift report<br>2. Inform Longwall Coordinator and Shift Supervisor of changes in conditions | 1. Note changed conditions on shift report<br>2. Inform Longwall Coordinator, Shift Supervisor, Longwall Superintendant, Geotechnical Engineer and Mine Manager |
| | **Shift Supervisor** | 1. Provide support to face supervisor<br>2. Communicate shiftly plan | 1. Inspect area during shift | 1. Inspect area ASAP<br>2. Inform Longwall Superintendant, Longwall Coordinator and Geotechnical Engineer of conditions | 1. Inspect area immediately<br>2. Report conditions to Longwall Superintendant and Geotechnical Engineer ASAP |
| | **Geologist** | 1. Map the longwall face once per week | | 1. Map face daily | 1. Map face daily |
| | **Geotechnical Engineer** | 1. Routine chock pressure and convergence monitoring<br>2. Interpret and issue Hazard Plans informing Longwall Superintendant of potential hazards | 1. Review Hazard Plan with Longwall Coordinator<br>2. If conditions continue for longer than 2 shifts conduct an inspection during the following shift | 1. Inspect area ASAP<br>2.Conduct daily inspection until conditions improve | 1. Visit affected area immediately |
| | **Longwall Coordinator** | 1. Ensure a leg and sprag audit is conducted prior to maintenance shift<br>2. Audit and monitor longwall standards | 1. Review Hazard Plan with Geotechnical Engineer<br>2. If conditions continue for longer than 2 shifts, conduct an inspection during the following shift<br>3. Inform Mine Manager of condition yellow if prevailing for longer than two shifts | 1. Inspect area as soon as possible | 1. Visit affected area immediately |
| | **Longwall Superintendant** | 1. Participate in the weekly Strata Control Review | 1. Arrange audit of emergency onsite recovery equipment | 1. Inspect area as soon as possible<br>2. Form interim plan with Longwall Coordinator and Geotechnical Engineer<br>3. Arrange availability of specialist contractors<br>4. Inform Mine Manager of conditions and actions | 1. Vist affected area immediately<br>2. Form Strata Management Team inclusing the Geotechnical Engineer and Longwall Coordinator as a minimum.<br>3. Mobilise required specialist contractors<br>4. Inform Mine Manager of conditions |
| | **Mine Manager** | 1. Approve TARP<br>2. Approve Hazard Plans | | | 1. Approve Strata Management Team plan of acton<br>2. Inform Senior Site Manager |

# Appendix 17: An Example of a Change Management Policy Pertaining to Ground Engineering

## Change Management Policy and Process Geotechnical Assessment

### Introduction

It is a requirement that, as part of the company's change management process, geotechnical assessment shall be undertaken during modification to the mine operation or design.

### Aim

The aim is to set the guidelines for the process of investigating the geological and geotechnical impact of changing any aspects of the mine operation.

### Scope

Potential changes to mine design or operation that may require assessment for geotechnical impact include:

(a) Ground support system
  (i) Equipment used in ground support (e.g. different continuous miner)
  (ii) Materials used in ground support (e.g. an alternative product, material or design)
  (iii) Installation methods (e.g. sequence of installation)
  (iv) Support plans and support rules (e.g. changing bolt density or placement)
(b) Procedures that may impact on ground support
(c) Major variation of the mine plan (e.g. changing roadway orientation, gateroad pillar size)
(d) Minor variations to mine plan (e.g. drivage dimensions or sequence)
(e) Introduction of technical advances (e.g. new products, installation methods)
(f) Different geotechnical assessment techniques and methods (e.g. new software or criteria for decision making).

All persons who have the authority to approve such changes to mine operations must ensure that any geotechnical impact is assessed.

### References or Related Documents

- Corporate Fatal Risk Control Procedures
- Geotechnical Risk Assessment Process (Corporate Standard Management Procedure No. ****)
- Risk Assessment Procedure (Corporate Standard Management Procedure No. ****)
- Mine Planning Management Procedure (Corporate Standard Management Procedure No ****)

### Verification and Auditing Guidelines

The verification that requirements of this procedure have been carried out will be evidenced by:

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(a) Geotechnical Engineer to keep "List of Geotechnical Assessments Completed" for changes to the mine design or operation requiring a geotechnical assessment.
(b) Geotechnical assessment reports, where relevant.
(c) Documented risk assessment, where required.

## Responsibilities

Mine Manager: confirm or sign-off that, where relevant, changes to any aspects of the mine operation have been assessed for geotechnical impact. Ensure that a risk assessment has been completed where required.

Geotechnical Engineer: to conduct an assessment of the geotechnical impact, in terms of hazards, risks and controls required. Initiate a risk assessment if the issue was not covered in a previous risk assessment, if it involves a physical change for the operations crews and if it is to be signed off by the Mine Manager. Maintain a list of changes assessed for geotechnical impact.

Longwall, Development and Services Coordinators: to recognise that changes to the mine operation may have a geotechnical impact and seek assessments from the Geotechnical Engineer.

Undermanager: to recognise that changes to the mine operation may have a geotechnical impact and report possible need of further assessment to Coordinators.

Mechanical and Electrical Engineer: recognise that changes to the mine operation may have a geotechnical impact and seek an assessment from the Geotechnical Engineer.

Mining Crews: confirm that changes to the mine operation are supported by a geotechnical assessment and a documented risk assessment (where required).

## Forms/Supporting Documents

- Maintain a "List of Geotechnical Assessments Completed" – in response to changes to mine design or operation.
- Geotechnical Hazard Identification Checklist.

## Procedure

(a) Procedure Flowchart (attached)
(b) Guidelines

The following guidelines indicate what level of geotechnical assessment may be required for operational changes. For each modification, the risk ranking of identified hazards should indicate what type of assessment of risk should be completed – as per Corporate Risk Assessment Standard.

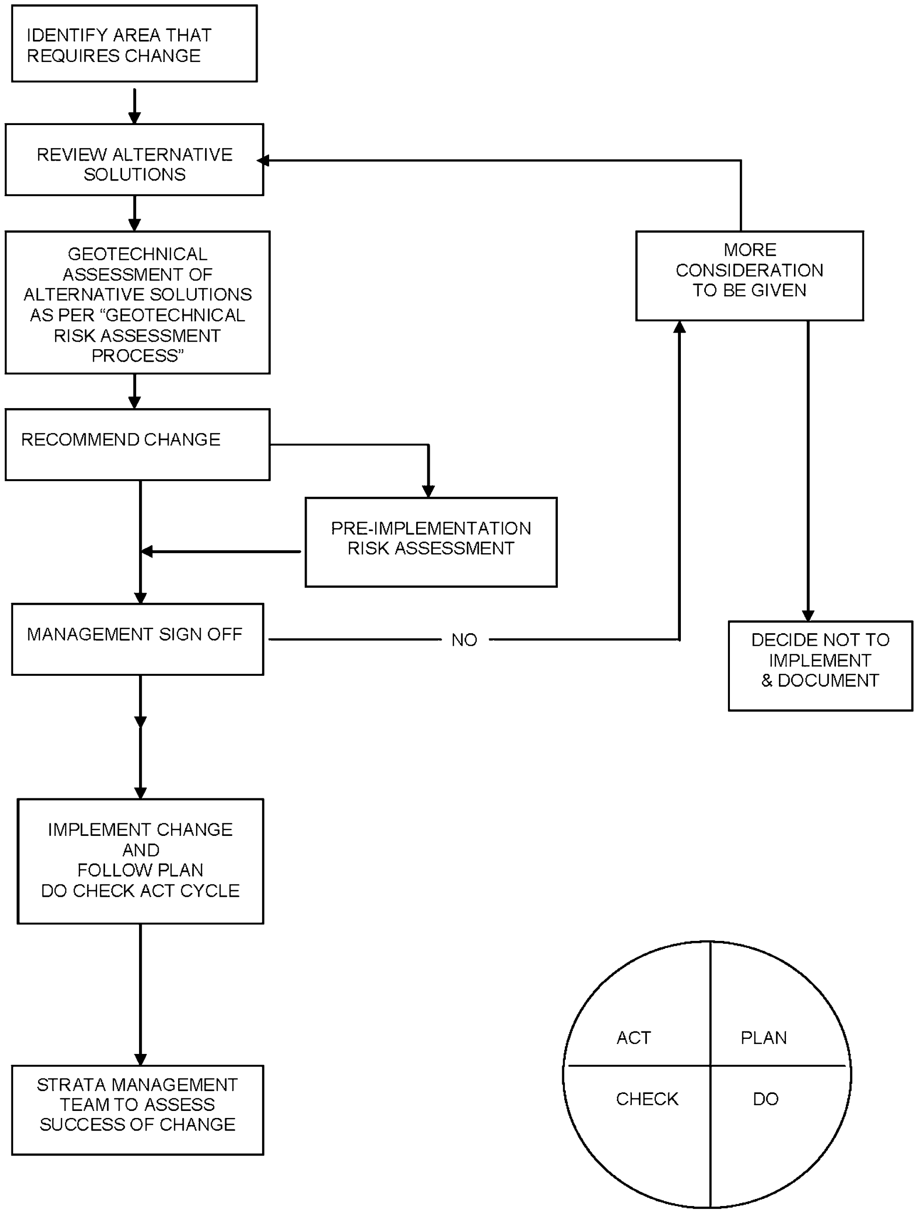
| Area of Modification | Guidelines | Geotechnical Assessment Report | Risk Assessment** |
|---|---|---|---|
| **1. Ground Support System**<br>i. Changes related to working panels may require a report and risk assessment.<br>ii. Changes to be implemented in future panels must be included in the pre-mining risk assessment for that panel. | | | |
| • Equipment used in ground support | Must meet the requirements of the support design specification, Strata Management Plan and Fatal Risk Procedures. | Likely | Yes |
| • Materials used in ground support | Must meet the requirements of the support design specification, Strata Management Plan and Fatal Risk Procedures. Assess properties of new materials. New materials may result in handling or installation modifications. | Yes | Yes – if a physical change |
| • Installation methods | Geotechnical assessment required for any modification. | Yes | Yes |
| • Support Plans and Support Rules | Geotechnical assessment and report for changes such as support density, bolt placement, longer bolts etc | Yes | Yes – if a significant physical change |
| • Ground conditions | If outside area of Trigger Action Response Plan a full geotechnical review and redesign is required | Yes | Yes |
| **2. Procedures that may impact on ground support.** | Assessment should be signed off | Yes | If required |
| **3. Variation of the Mine Plan** | Geotechnical Engineer notified of variation according to the "Mine Plan Management Procedure" (Doc No.****) | If required | If required |
| **4. Minor operational variations to the Mine Plan** | Operations Co-ordinators to advise geotechnical engineer on changes such as mining sequence, roadway dimensions, adjustments to bolting cycle etc. | Yes | Possible |
| **5. Introduction of technical advances** | A full geotechnical assessment of new methods, systems or technical improvements. | Yes | Yes |
| **6. Different geotechnical assessment techniques** | Validate against existing methods. | Yes | In pre-mining risk assessment |

**Level of Risk Assessment to be determined using the Corporate Risk Assessment Standard

## Procedure Review

The effectiveness of this procedure will be reviewed every 2 years by the Mine Manager. It will also be reviewed at any time it is shown to be ineffective.

# GROUND CONTROL MANAGEMENT PLAN CHANGE MANAGEMENT PROCEDURE


IDENTIFY AREA THAT REQUIRES CHANGE
REVIEW ALTERNATIVE SOLUTIONS
GEOTECHNICAL ASSESSMENT OF ALTERNATIVE SOLUTIONS AS PER "GEOTECHNICAL RISK ASSESSMENT PROCESS"
RECOMMEND CHANGE
PRE-IMPLEMENTATION RISK ASSESSMENT
MANAGEMENT SIGN OFF
NO
MORE CONSIDERATION TO BE GIVEN
DECIDE NOT TO IMPLEMENT & DOCUMENT
IMPLEMENT CHANGE AND FOLLOW PLAN DO CHECK ACT CYCLE
STRATA MANAGEMENT TEAM TO ASSESS SUCCESS OF CHANGE
ACT
PLAN
CHECK
DO

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# Appendix 18: An Example of a Ground Control Monitoring Plan Procedure

## Ground Control Monitoring Plan Procedure

**Note**
This example of a Ground Control Monitoring Plan Procedure is presented to illustrate the concepts associated with such a plan. It is not endorsed as being completely robust or universally applicable.

### Introduction

Legislation, company policies and standards and the Ground Control Management Plan have particular monitoring requirements.

This procedure provides guidelines for establishing the monitoring requirements for 5 different areas of ground control management. These areas are:

(a) Ground control monitoring for each panel. This principally covers ground deformation – comparing actual and expected.
(b) Monitoring the quality of ground support installation.
(c) Monitoring ground support equipment
(d) Monitoring quality control of ground support materials
(e) Assessment of monitoring data.

### Aim

This procedure sets the guidelines for establishing a comprehensive monitoring plan for all aspects of ground control.

### Scope

A monitoring plan is to be developed by the Geotechnical Engineer for each mining panel. It may also apply to site-specific support design (e.g. drivehead installation sites etc.). Table A18.1 provides information on the application of monitoring with respect to:

(a) Timing
(b) Nature of the monitoring
(c) Allocation of responsibility
(d) Nature of reporting

### References or Related Documents

- Legislative requirements
- Company policies and standards
- Procedure for Ground Control Monitoring – Part 2

### Training

Training for monitoring according to the TARP will be provided to mining crews at the commencement of each mining panel.

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**Table A18.1** Schedule for monitoring ground support requirements

| | Ground control & displacement monitoring | Ground support installation quality | Ground support equipment | Ground support materials | Assessment of monitoring |
|---|---|---|---|---|---|
| Shift/ Daily | Deputies and Undermanagers: | Deputies and Undermanagers: | Mech Eng, Deputies and Undermanagers: | Deputies and Undermanagers: | Deputies: |
| | Inspections | Inspections | Inspections | Inspections | Observe, read and report as required |
| | Shift reports | Shift reports | Shift reports | Shift reports | Assess against trigger levels |
| Weekly | Geotechnical engineer: | Geotechnical engineer: | Geotechnical engineer: | Geotechnical engineer: | Geotechnical engineer: |
| | Review | Review | Review | Review | Database up-to-date |
| | Report by exception | Report by exception | Report by exception | Report by exception | Report by exception |
| Monthly | Geotechnical engineer: | Geotechnical engineer: | Geotechnical engineer: | Geotechnical engineer: | Geotechnical engineer: |
| | Maintain monitoring schedules | Check bolt/support installation process | Verify inspections and tests done | Receive batch test results from suppliers | Confirm support design |
| | | Report by exception | Report by exception | Report by exception | Prepare monthly report |
| 12 monthly | Geotechnical engineer: | Geotechnical engineer: | Geotechnical engineer: | Geotechnical engineer: | |
| | Arrange audit of procedures | Check entire mine site strata support process from supply of materials to installation | Check status of testing system and equipment issues | Audit checklist for suppliers | |
| | Prepare audit report | Report | Report | Arrange independent tests of materials | |
| | | | | Report | |

A monthly status report on all monitoring is required from the Geotechnical Engineer

The Undermanagers and Geotechnical Engineer will be trained in their monitoring responsibilities during induction; monitoring will be part of their role description.

## Procedure Review

The effectiveness of this procedure will be reviewed each 2 years by the Mine Manager. It will also be reviewed at any time it is shown to be ineffective.

## Procedure – The Panel Monitoring Plan

(a) Aim
Provide guidelines for writing and planning a specific monitoring plan for each panel.
(b) Verification and Auditing Guidelines

  - (i) The verification that requirements of this procedure have been carried out will be evidenced by:
  - (ii) Monitoring that is specified in each TARP is supported by a documented Monitoring Plan.
  - (iii) Monitoring reports are provided as required by TARP.
  - (iv) Monitoring results are available on site (underground) with suitable monitoring tools (Tell Tales and Gels).
  - (v) Monitoring database is up-to-date.

    Note: System Auditor would audit items (a) and (d); Undermanager to audit items (b) (shiftly) and (c) (weekly).

(c) Responsibilities

Geotechnical Engineer: to provide a monitoring plan for each TARP, a maintenance schedule for monitoring apparatus, and complete their own tasks within the monitoring schedule.

Undermanager: to audit the shift reports for ground control information, and confirm that monitoring apparatus are active and monitoring results are available on site.

Mining Crews: to monitor and report ground conditions according to the TARP.

(d) Forms/Supporting Documents

  - (i) Monitoring Plan (report)
  - (ii) Monitoring result sheets (may be part of shift report)
  - (iii) Monitoring schedule (may be on TARP)
  - (iv) Maintenance schedule for monitoring apparatus
  - (v) Mine plan with monitoring points and status showing if active or inactive

(e) Standards

The following standards apply to the development of the panel ground control monitoring plan:

  - (i) Ground control monitoring will measure the amount and rate of ground movement at both unique (geotechnical) sites, and appropriate representative sites in a mining panel. It will also observe visual deformation of ground conditions and support.
  - (ii) Monitoring requirements should be based on the expected or known geotechnical variation in each panel. For example, variations in geology, geotechnical properties, and changes in ground support may define a geotechnical domain. Different levels of confidence in the knowledge of, or potential impact of, any of these should be considered.
  - (iii) Monitoring generally should focus on providing data to assess changing conditions or changed ground support.
  - (iv) Monitoring requirements for any geotechnical domain should define:
    1. type of instrument, or type of monitoring
    2. location
    3. reading frequency
    4. reporting methods
    5. geotechnical mapping activities
    6. responsibility for inspection, testing, installation, reading and reporting.
  - (v) Monitoring requirements are generally included in the TARP for development, longwall or outbye roadway areas. They are linked to the expected geotechnical conditions:
    1. trigger levels (for example, displacement rate, total displacement and longwall acceleration position).
    2. visual signs.
  - (vi) Specific monitoring instructions may be issued, in addition to those included in the TARP.
  - (vii) A mine plan should be maintained by the Geotechnical Engineer that shows the location and status of each monitoring zone and each monitoring apparatus.
  - (viii) Each monitoring apparatus (of appropriate type) is to be maintained so that:

1. Inactive monitoring points are removed or tagged.
2. Damaged monitoring points are replaced if still required.
3. Monitoring data is recorded and maintained at the site to allow persons to immediately assess ground movement status.
4. A maintenance schedule is used to confirm that monitoring apparatus are "active".

## Procedure – Monitoring the Quality of Ground Support Installation

(a) Aim

This procedure is concerned with verifying that the systems, procedures, training, equipment and methods used result in the effective installation of ground support materials.

(b) Verification and Auditing Guidelines

The verification that requirements of this procedure have been carried out will be evidenced by:

- (i) Shift reports that note compliance & non compliance with the Support Rules, Support Plans and TARP.
- (ii) Report by the Geotechnical Engineer as required. This includes a regular status report to the Strata Management Team.

(c) Responsibilities

Geotechnical Engineer: this role has the responsibility to co-ordinate the monthly auditing of ground support installation quality. Reports of each audit will be kept and reported to the Strata Management Team meetings. The Geotechnical Engineer is responsible for co-ordination of any required corrective action.

(d) Forms and Supporting Documents

- (i) Support Rules, Support Plans and TARPs
- (ii) Mine plan with monitoring points and their status
- (iii) Checklists for Monthly Audits – Appendix 1
- (iv) Schedule of Ground Support Materials, Suppliers, Contacts etc. – Appendix 2
- (v) Installation testing procedures

(e) Monitoring Schedule

The schedule for monitoring ground support installation quality is:

- (i) Shiftly and Daily inspections by mining crews and statutory officials. Inspect compliance with the Support Rules and TARPs.
- (ii) Weekly: the Geotechnical Engineer inspects each working area and reports by exception. Inspect compliance with the Support Rules and TARPs.
- (iii) Monthly: the ground support installation process is audited by checklist.
- (iv) 12 Monthly: a full audit of the ground support installation process

(f) Standards for Audit

(i) Shiftly and Daily

The specification of ground support requirements are provided in the Support Rules, Support Plans, and TARP. Items that might be included in monitoring ground support installation are:

1. Compliance with Support Rules etc.
2. Bolt spacing, row spacing
3. Position of bolts, mesh
4. Distance from last support to face
5. Bolt tail length
6. Spin times
7. Bolt encapsulation.
8. The monitoring is reported in shift reports.

(ii) Weekly

Each week the Geotechnical Engineer will inspect working panels for compliance to support rules, support plans and TARPs. This will also include assessment of information in shift reports reported by exception.

(iii) Monthly

Each month the Geotechnical Engineer shall complete an audit of a panel to check the bolt installation process and check that the equipment being used is running to correct specifications. This audit may be complemented by supplier audits or checks. All support types in the panel should be audited. Audit to include:

1. Bolt/support type in use
2. Crew on shift
3. Other installation hardware (mesh, straps, plates)
4. Batch numbers
5. Chemical type
6. Drill bit size
7. Resulting hole diameter
8. Hole length
9. Spin and hold times
10. Encapsulation length in hole
11. Torque settings on rig
12. Bolt torque.

(iv) 12 Monthly

The purpose of this audit is to check the entire strata support process from supply of the support materials to the completion of the installation underground. These audits are to be carried out by a team comprising at least 2 people, one of whom will be the Geotechnical Engineer. The other person(s) on this team (nominated by the Geotechnical Engineer), will have expertise in a particular area of the strata support process, such as,

1. Production manager
2. Team leader
3. Operator
4. Consumables supplier
5. Mechanical engineer or tradesman
6. Geotechnical expert

In addition to the items checked in daily, weekly and monthly audits the following are noted in 12 monthly audits:

1. Storage of consumables on surface
   (a) Stock
   (b) Condition
   (c) Use by dates
2. Bolt storage
3. Resin storage
4. Transport underground
5. Storage underground
6. Pull out tests
7. Drill steel length
8. Position of installed bolts versus approved pattern
9. Maintenance of bolting equipment.
10. Use of testing procedures

(g) Reporting and Corrective Action

Problems with support installations observed on shift are reported in shift reports. Corrective actions are taken at the time, TARPs may be used, or work stopped until the problem is resolved.

The Geotechnical Engineer is to report monthly on the quality of support installation as required and on the monitoring results. The Geotechnical Engineer is responsible for co-ordination of any required corrective action.

## Procedure – Monitoring Ground Support Equipment

(a) Aim

To provide communication if defective equipment is involved with the ground support process to allow correction and assessment of potential impact.

(b) Verification and Auditing Guidelines

The verification that requirements of this procedure have been carried out will be evidenced by:

(i) Maintenance reports.
(ii) Monthly monitoring report by the Geotechnical Engineer to include "by exception" reports of defective equipment used for ground control.
(iii) 12 monthly audit reports.

(c) Responsibilities

Mechanical Engineer: is responsible for the maintenance programs and maintenance schedule for equipment, including ground support equipment. Ground Support Equipment that is operating out of specification should be notified to the Process Co-ordinator and the Geotechnical Engineer to judge its impact on the effectiveness of ground support.

Geotechnical Engineer: To maintain a checklist of the ground support equipment in use. To investigate "by exception" reports of defective ground support equipment. Assess the impact of equipment not operating to standard. Reports are to be completed as required.

(d) Forms and Supporting Documents

Checklist of Ground Support Equipment and Maintenance Schedule – Appendix 3.

(e) Monitoring Standards

- (i) Shiftly/Daily/Weekly
  A maintenance schedule is available on ground support installation equipment, and longwall supports. Inspections made by tradesmen/engineers according to the inspection schedule. Mechanical Engineer to notify Geotechnical Engineer if ground support equipment is not operating to specification.
- (ii) Geotechnical Engineer to have a checklist of such equipment and maintenance schedule. Refer to Appendix 3.
- (iii) Weekly
  The Geotechnical Engineer to review impact of equipment not operating to specification
- (iv) Monthly
  The Geotechnical Engineer reviews the status of ground support equipment. Follow up on reported defective equipment and progress or corrective action. Report by exception.
- (v) 12 Monthly
  Audit status of inspection system, equipment issues and corrective action. Report audit findings.

(f) Reporting and Corrective Action

The maintenance and inspection system will report and schedule corrective action for equipment.

The Geotechnical Engineer monitors results to confirm the integrity of ground support. Consequently reports that include the status of ground support equipment is reported by exception. Monthly status reviews are reported by exception and 12 monthly system audits reports are completed. Reports are to be made to the Strata Management Team. Immediate corrective action is directed by the TARP. The Mechanical Engineer is responsible for co-ordination of any required corrective action.

## Procedure – Monitoring Quality of Ground Support Materials

(a) Aim

To provide methods for confirming that the ground support materials, as they are supplied to the mine, fit the specifications required in the ground support design.

(b) Verification and Auditing Guidelines

The verification that requirements of this procedure have been carried out will be evidenced by:

- (i) Test reports from Suppliers
- (ii) Monthly monitoring checklist by the Geotechnical Engineer
- (iii) 12 Monthly monitoring report by the Geotechnical Engineer.

(c) Responsibilities

Geotechnical Engineer: to co-ordinate the monitoring program of ground support materials supplied to the Mine. Reports shall be supplied to the Strata Management Team for:

- (i) monthly test reports are received and accepted, or rejected; and
- (ii) following 12 monthly audits.

(d) Forms and Supporting Documents

- List of ground support suppliers, contact details, materials and manufacturers materials specifications. Appendix 2
- Current ground support materials specifications – Appendix 4

- Monthly Test Checklist for each Supplier – Appendix 5
- 12 Monthly Test Checklist for each Supplier – Appendix 6
- Test Procedures – Appendix 7

(e) Monitoring Schedule

The monitoring schedule is as follows:

1. Shiftly, daily and weekly. Reports by exception from use and installation of materials.
2. Monthly: Provision and review of Supplier test results for batches delivered to site.
3. 12 Monthly: Detailed review of part of each suppliers quality control system including visits to suppliers sites. Includes testing of materials.

(f) Monitoring Standards

(i) Shiftly/Daily/Weekly

During the handling, storage and installation of ground support materials any abnormality to materials should be recorded on shift reports. Suppliers are to be notified if materials are being supplied contrary to normal specification.

Such items may include:

1. Different shape, size, or design
2. Different handling arrangements, or packaging
3. Different installation behaviour
4. Materials not “fit-for-purpose”

(ii) Monthly

Geotechnical Engineer to receive manufacturing test reports on recent batches delivered by the Supplier. These should be reviewed and confirmed to fit product specifications. Appendix 4 contains a checklist of quality reports which should be provided by the Supplier each month.

(iii) 12 Monthly

Each 12 months a detailed audit program will be conducted on the Suppliers of all ground support products.

This will include:

1. Review the Suppliers current materials specifications, compare with contract or supply agreement.
2. Review the Mines current support design specifications
3. Review the Suppliers testing procedure to confirm product specifications.
4. Testing an example of each ground support apparatus to confirm that it meets Suppliers specifications.
5. Review manufacturing quality control system – confirm that it is able to supply materials to specification. Understand the criteria used for triggering rejection of product.
6. Review Suppliers installation procedure – ensure it is included in mine procedures
7. Review training information supplied by Supplier
8. Review supply system to mine.

Facilitate this by using a 12 Monthly Test Checklist (Appendix 5) that includes all ground support items from all Suppliers.

(g) Reporting and Corrective Action

Geotechnical Engineer should report monthly on the quality of materials delivered to the mine site during the last month if they are out of specification. If materials are found to be out of specification the Geotechnical Engineer is to co-ordinate corrective action. Each 12 months the Geotechnical Engineer will report the results of tests conducted on the ground support materials and review of the Suppliers systems.

## Procedure – Assessment of Monitoring Data

(a) Aim

To ensure that adequate assessment is made of the range of monitoring data collected.

Assessment is to provide a more detailed understanding of the adequacy of the support design. Different monitoring programs aim to identify elements that may limit support effectiveness.

(b) Verification and Auditing Guidelines
The verification that requirements of this procedure have been carried out will be evidenced by:
  (i) Updated monitoring database, including ground deformation monitoring
  (ii) Monthly monitoring assessment report by the Geotechnical Engineer

(c) Responsibilities
The Geotechnical Engineer is to evaluate the monitoring data, or co-ordinate expert assistance, to assess the effectiveness of the ground support system.

(d) Standards of Assessment
  (i) Shiftly/Daily
  Monitoring data is collected and reported according to TARP or monitoring schedules controlled by the Geotechnical Engineer. In all cases the results are tested against the TARP trigger levels.
  (ii) Weekly
  Monitoring data (ground displacement) is to be updated into the data base at least weekly (having been tested against the trigger levels on the day of reading).
  (iii) Monthly
  A report is made that uses the available monitoring data to confirm the ground support design. Monitoring data of support installation, ground support equipment and ground support materials should be utilised in a full evaluation.

(e) Reporting
The reporting requirements are detailed in the individual sections of this procedure. Observations that are made on a routine shift or daily basis are reported in shift reports. The Geotechnical Engineer is required to complete Monthly monitoring reports. These monitoring reports should be a consolidated report for the reporting requirements outlined in the different sections of this procedure.

**To complete this procedure the Appendices need to be completed. They provide the detail and substance of this procedure. Some liaison with suppliers will be required to complete Appendices 4, 5, 6, and 7.**

Appendix 1: Checklist for Quality of Installation (Monthly)

Appendix 2: Schedule of Ground Support Materials, Suppliers, Contact Details and Manufacturers Materials Specifications

Appendix 3: Checklist of Ground Control Installation Equipment

Appendix 4: Current Ground Support Materials Specifications.

Appendix 5: Supplier Quality Control Test Report checklist

Appendix 6: 12 Monthly Test Checklist for each Supplier

Appendix 7: Test Procedures for Ground Support Materials

# Glossary of Terms

| | |
|---|---|
| Abutment | The zone of unmined rock around the perimeter of an excavation. |
| Act | A law made by government. |
| Adit | An entry driven in coal from a point where the coal seam outcrops on the surface. |
| AFC | Armoured face conveyor – the chain conveyor installed on a longwall face. |
| ALARP | As low as reasonably practical. |
| ALPS | Analysis of Longwall Pillar Stability – an empirical pillar design methodology for longwall mining. |
| ALTS | Analysis of Longwall Tailgate Serviceability – an empirical pillar design methodology for longwall mining. |
| Angle of Draw | Defines the lateral extent of mining-induced vertical displacement on the surface. It is the angle between two lines drawn from the edge of the mine workings, one a vertical line and the other a line to the limit of vertical displacement on the surface. |
| Anisotropic | Having different physical properties in different directions. An anisotropic material reacts differently in different directions to the same applied stress. |
| Aquiclude | A body of rock which is effectively impermeable. |
| Aquifer | A permeable body of rock or regolith that both stores and transmits water (DoP 2010b). |
| Aquitard | A body of rock which has a very low permeability, sufficient to significantly impede the transmission of water. |
| ARMPS | Analysis of Retreat Mining Pillar Stability – an empirical pillar design methodology for pillar extraction mining. |
| Bag | A colloquial term for ventilation sheeting, or brattice, used to partition a roadway into an intake and a return airway in order to direct air to the coal face. |
| Barring Down | The act of prising loose pieces of rock from the roof and ribsides using a bar. Also referred to as 'scaling down'. |
| Baulk | A wooden cross support with a round (log), half-round (split log) or square (milled log) cross-section. |
| Bedding Plane Shear | Shear displacement along a bedding plane. |
| Bi-directional cutting (Bi-di) | The process of cutting the longwall face to its full height in a single pass of the shearer, both from the maingate to the tailgate end and the tailgate to the maingate end. |
| Bleeder Roadway | A return airway that flanks a mining panel or a series of mining panels for the purpose of promoting a flow (bleed) of air through completed mine workings or goaves. |
| Boot-end | The return roller end of a conveyor belt. This is the loading point in a production panel. |

(continued)

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| | |
|---|---|
| Bounce | See ‘pressure bump’. |
| Brat | See ‘scat’. |
| Breaker Prop | A prop, usually of timber, set for the purpose of breaking off a fall of roof so as to prevent it from overrunning into the workplace. |
| Breaker Line | A series of closely spaced rows of breaker props which act as a fulcrum to break off a fall of ground to prevent it from entering the workplace. Typically, each row comprises four to six breaker props, and each breaker line comprises two or three rows. |
| Brow | The free, or cantilevered, edge of a step in the immediate roof resulting from a change in mining horizon. May also be referred to as a ‘lip’. |
| Brownfield | A geographical area in which there is a history of coal mining operations. |
| Brush | To increase the height of a roadway by mining additional material from the roof. |
| Bump | See ‘pressure bump’. |
| Burst | See ‘pressure burst’. |
| Clacking | A colloquial term that refers to the noise made by pressure relief valves when a powered support yields by releasing hydraulic fluid. |
| Cleat | A natural system of joints, or cleavage, within a coal seam. Cleat is usually comprised of two conjugate joint sets that are perpendicular or near perpendicular to stratification. It is often confined to specific coal plies. One joint set is usually more dominant and is referred to as face cleat; the other joint set is known as butt cleat. |
| Competent | Possessing sufficient knowledge, skill and experience to perform a function or task to an acceptable standard. |
| Consequence | With respect to risk: Outcome of an event affecting objectives (ISO 31000 2009).<br>–<br>With respect to ground subsidence: Any change in the amenity, function or risk profile of a natural or man-made feature due to the impact of ground subsidence. |
| Contact | The plane or surface where two different rock types meet. |
| Control | A process, policy, device, practice or other action which modifies risk (ISO 31000 2009). A control can act to minimise negative risk or enhance positive opportunities. |
| Convergence | The elastic rebound of the rock mass into an excavation due to removal of the virgin stresses from the surface of the excavation. At depth, the magnitude of these stresses is such that the amount of convergence cannot be restrained by any practical form of artificial support (Jaeger and Cook 1979). |
| Cutter | Another term for ‘guttering’. |
| Dip | The angle at which a bed, stratum, or vein is inclined from the horizontal, measured perpendicular to the strike in the vertical plane.<br>Also referred to as ‘hade’. |
| Discontinuity | A mechanical break in the fabric of the rock mass across which there may or may not have been relative displacement. Discontinuities include fault planes, dykes, joints and bedding planes. |
| Dripper | A joint plane, crack or drill hole that drips water from the roof of mine workings. |
| Drummy | Refers to a situation where one or more parting planes are present in the immediate roof strata. When struck with a steel, the strata sounds hollow or ‘drummy’, as opposed to ‘ringing’ sharply. |
| Dyke | A near vertical intrusion of igneous rock. |
| Empirical | Based on observation or experiment. |
| Engineering | Is the knowledge required and the process applied to conceive, design, make, build, sustain, recycle or retire, something of significant technical content for a specified purpose (Brown 2001a). |

(continued)

| | |
|---|---|
| Event | An occurrence or change of a particular set of circumstances (ISO 31000 2009). |
| Face break | The snapping off of the immediate roof at the face line in longwall mining. |
| Fault | Geological – A planar discontinuity between blocks of rock along which relative shear displacement has occurred. |
| Feather edge | A term describing a type of roof fall in which the roof falls as a thin wafer of rock, tapering back from around 0.5–1.0 m in thickness to almost a razor sharp edge in many cases. These low angle shears are mostly associated with brittle strata, particularly sandstone and conglomerate. |
| Fender | A long rectangular or slender web of coal separating a split or lift from the goaf. Also referred to as a 'wing' or a 'web' in some situations. A fender may or may not be subsequently partially or totally extracted. |
| Flit | A term used to describe the process of relocating mobile face equipment in an underground coal mine. Most often applied to the relocation, or tramming, of a continuous miner. |
| FMRS | A Fletcher Mobile Roof Support – a form of mobile roof support (MRS) manufactured by Fletcher. |
| Footwall | The floor or base of a mine opening. A footwall is not necessarily horizontal and is distinguished from the hanging wall in that gravity acts to keep the rock mass in position. In a vein or bedded deposit, the footwall may comprise the top surface of the rock stratum underlying the deposit. |
| FOG | Fall of ground. |
| Fracture | A natural or mining induced planar discontinuity between blocks of rock along which extremely little or no discernible displacement has occurred. |
| Fracturing | The formation of planes of separation in the rock material, involving the breaking of bonds to form new surfaces. The onset of fracture is not necessarily synonymous with failure or with the attainment of peak strength (Brady and Brown 2006). |
| Fretting | The weathering/spalling/disintegration of the ribs or roof in small pieces over a period of time. |
| Friable | Easily broken. |
| Gas Content | The total desorbable volume of gas contained in a known mass of coal at in-situ conditions expressed in cubic metres per tonne of coal at 20 °C and 101.3 kPa. |
| Gate-end | That area where the end of a longwall face intersects a gateroad. |
| Gateroad | A roadway that flanks the length of a longwall panel. |
| GCMP | Ground Control Management Plan. |
| Geological Structure | Refers to all natural planes of weakness in the rock mass that pre-date any mining activity and includes: joints, faults, shears, bedding planes, foliation and schistosity (NSW Dept. Mineral Resources 2004). |
| | – |
| | Any disturbance whereby a coal seam is altered from its original depositional state. |
| Geomechanics | Is concerned with the physical and mechanical properties and responses of soils and rocks and their interactions with water and encompasses the subject of rock mechanics (Brown 1998). |
| Gob | Another term for 'goaf'. |
| Goaf | An area in which mining has been completed and left in a partially or totally collapsed state or in an inadequately supported state to assure safe entry. An abandoned area. Also referred to as 'gob'. |
| Goaves | Plural of 'goaf'. |

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| Greasy back | A slickensided surface within the immediate roof strata. |
| Greenfield | A geographical area in which there is no previous history of coal mining operations. |
| Grub | To increase roadway height by going back and excavating more material from the floor. |
| Grunch | The process of shot-firing a coal face without first forming a second free face by cutting a slot in the face. |
| Ground Control | A term used more commonly in mining than in the civil construction industry and taken to mean the maintenance of the stability of the rock around an excavation and the more general control of displacements in the near-field of an excavation (Brady and Brown,1993). |
| Guideline | A principle, criterion or advice intended to set direction and standards. |
| Guttering | Shearing of the roof resulting in a steep sided channel. Usually occurs in the roof/rib corner section of a roadway. |
| Hanging wall | The roof or top of a mine opening. A hanging wall is not necessarily horizontal and is distinguished from the footwall in that it is undercut and, therefore, subject to material being dislodged from it under the influence of gravity. In a vein or bedded deposit, the hanging wall may comprise the bottom surface of the rock stratum overlying the deposit. |
| Hang-up | A situation in which the intended caving of the roof strata has not occurred. |
| Hazard | A source of potential harm or a situation with a potential to cause a loss (including to people, property, the natural environment, business, or reputation). |
| Heterogeneous | Of non-uniform composition. |
| Homogenous | Of uniform composition. Homogeneity is a measure of the physical continuity of the rock mass based on the distribution of discontinuities and pore space within the window of interest. |
| Hydromechanical Couple | Refers to the physical interaction between hydraulic and mechanical processes. |
| Hypothesis | A proposition or supposition made as a basis for reasoning without reference to its truth. |
| Immediate Roof | The nether roof of the mine workings, defined to extend to various heights above the mine workings roof, typically ranging from 10 m to ten times the mining height. |
| Inbye | In a direction into the mine; in the direction of the working face. |
| Inundation | An inflow of fluid (in a gaseous or liquid phase) or other material that develops over a period of time sufficient for it not to present an immediate risk to health and safety. |
| Inrush | A sudden and unplanned or uncontrolled inflow into mine workings of fluid (in a gaseous or liquid phase) or other material that has the potential to result in unacceptable risk to health and safety. |
| Intact Rock | Rock which contains no discontinuities. |
| Joint | A natural planar discontinuity between blocks of rock along which little or no discernible displacement has occurred. Joints which are parallel in dip and strike over a considerable area constitute a joint set. Two or more joint sets that intersect at more or less a constant angle constitute a joint system. |
| Isotropic | Having the same physical properties and, therefore, the same reaction to applied stress in all directions. |
| Lacing | A pattern of cables strung between tendons to aid in confining intermediate strata and screen supports. Also used to provide a yielding capacity in environments susceptible to pressure bursts. |
| Lagging | Timber or steel used to infill between roadway supports to prevent the ingress of rock. |

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| Laminations | Layers within beds of strata. |
| Lift | A slice of coal mined from a pillar for the purpose of extracting the pillar. A lift may be mined from a heading, cut-through or split. |
| Likelihood | Chance of something happening (ISO 31000 2009). |
| | – |
| | Probability or frequency of an event occurring. |
| Lineament | A topographic alignment of features that appears to be structurally controlled. Also referred to as a ‘fracture trace’ or ‘photolineament’. |
| Lip | The edge of a brow, or step, in the profile of the immediate roof. A lip may be the result of a step change in mining horizon or a fall of ground. |
| Lithology | The character of the rock described in terms of its structure, colour, mineral composition, grain size, and arrangement of component parts (Gates et al. 2008). |
| Loss | Any negative consequence. |
| LTIFR | Lost Time Injury Frequency Rate – lost time injuries per one million hours worked. |
| Massive | In geology, the term is used to describe a rock mass that has a paucity of well developed bedding planes. |
| MBLS | Mobile Breaker Line Support, a form of mobile roof support (MRS) developed by Voest Alpine. |
| Mining Geomechanics | Is that part of geomechanics (including rock mechanics) that is concerned with the application of knowledge of the physical and mechanical behaviour of geological materials (soils, rock and water) to the investigation, design, operation and performance of mining structures including excavations (Brown 1998). |
| Monitor | To check, supervise, observe critically or measure the progress of an activity, action or system on a regular basis in order to identify change from the performance levels required or expected (AS/NZS-4360:2004 2004). |
| MRS | Mobile roof support, based on a powered longwall support. |
| MSHA | (USA) Mine Safety and Health Administration. |
| Muck | (noun) – a pile of broken rock, usually resulting from blasting or a fall of ground. |
| | (verb) – the action of relocating a pile of broken rock. |
| NCB | (British) National Coal Board. |
| NIOSH | (USA) National Institute of Occupational Safety and Health. |
| Normal Fault | A fault plane in which the direction of movement results in the extension of stratum across the fault plane. |
| Organisation | A group of people and facilities with an arrangement of responsibilities, authorities and relationships (AS/NZS-4360:2004 2004). |
| Orthogonal | At right angles to. |
| Outburst | A phenomenon in which a high concentration of gas usually accompanied by coal is expelled from the roof, floor or sides of a coal mining face. Disturbance is confined to the coal seam and occurs when the pressure of the desorbed gas within the seam exceeds the confinement provided by the rock mass, resulting in an inrush or inflow of material as a fluidised bed propelled by the desorbed gas. |
| | In the USA, the term is sometimes used to refer to a pressure burst. |
| Outbye | In a direction out of the mine; in the direction of the surface. |
| Overburden | A generic term encompassing all solid and liquid material overlying a mine. |
| Overcast | A construction at the intersection of two roadways in an underground ventilation circuit which permits intake air in one roadway to cross over return air in the other roadway. |
| Parallelepiped | A polyhedron comprised of six faces that are parallelograms. |
| Parting | A mechanical weakness within bedding comprised of a lamina or thin bed of weak material which may vary in thickness from a fraction of a millimetre to some tens of millimetres. |

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| | The weak material promotes separation of the strata (adapted from Cook et al. 1974). |
| | – |
| | An opening due to separation between bedding planes. |
| Permeability | A measure of the rate at which fluid can be transmitted through a body. |
| Piezometer | A non-pumping well or borehole, generally of a small diameter, used to measure the elevation of the water table or potentiometric surface (DoP 2010b). |
| Pillar Point | Created when a pillar is adjacent to extensive mined-out goaf on two sides (NIOSH 2010). Also referred to as an ‘arrow head’. |
| Pillar Stripping | The process of reducing the size of a pillar by mining lifts from its perimeter. Also referred to as ‘slabbing’. |
| Plunge | The distance that a roadway is advanced between ground support cycles; the ‘cut-out’ distance. |
| Pocket | A blind lift separated from an adjacent lift by a narrow fender or web of coal. |
| Policy | A course or principle of action or behaviour decided by government, management or individuals. |
| Pot Arse | A block of roof material, often dome shaped, that is defined by slickensided or smooth contact surfaces of negligible tensile strength. |
| Pozzolanic | Possessing natural self-cementing properties. |
| Pressure Bounce | A heavy, sudden, often noisy blow or thump; sudden spalling off to sides of ribs and pillars due to excessive pressure; any dull, hollow, or thumping sound produced by movement or fracturing of strata as a result of mining operations. Also known as a ‘bump’ (Gates et al. 2008). |
| Pressure Bump | Defined in this text as a seismic event that can be felt by the human body but does not result in ejection of material into the workplace. |
| | An event associated with the dynamic release of energy within the rock mass that is of sufficient magnitude to generate an audible signal; ground vibration; and potential for displacement of loose or fractured material into the mine workings. Also referred to in the USA as a ‘bounce’. |
| Pressure Burst | Defined in this text as a pressure bump that results in dynamic rock failure (including coal) in the vicinity of a mining excavation, resulting in high velocity expulsion of the failed material into the excavation. The energy levels and associated velocities are sufficiently high to result in significant damage to, and even destruction of, conventional rock mass support and reinforcement systems. |
| | An explosive breaking of coal or rock in a mine due to pressure; the sudden and violent failure of overstressed rock resulting in the instantaneous release of large amounts of accumulated energy where coal or rock is suddenly expelled from failed pillars (Gates et al. 2008). |
| | Can also be referred to in the USA as ‘bump’ or as a ‘bounce’. |
| Primary Workings | Workings driven in the process of developing the main arteries of a mine. |
| Principal Hazard | A hazard with the potential to cause multiple fatalities. |
| Procedure | A series of steps and actions carried out in a certain order or manner. |
| Process | A set of interrelated resources and activities which transform inputs into identifiable outputs. |
| Pushout | A USA term for a stook or stump formed in the vicinity of an intersection when extracting a coal pillar. |
| Regolith | The blanket of soil and loose rock fragments overlying |

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| | bedrock. It includes dust; soil; broken and weathered rock; and other related materials (DoP 2010b). |
| Regulation | Subordinate legislation in support of an Act. |
| Reinforcement | Measures which act from within the rock mass to improve the overall rock mass properties. Examples include rock bolts, cable bolts and strata binders (adapted from Brown 1998). |
| Residual Risk | The risk remaining after risk treatment (ISO 31000 2009). |
| Retreat Mining | A mining process in which secondary extraction is undertaken as mining operations retreat out of a panel under the protection of the primary workings. |
| Reverse Fault | A fault plane in which the direction of movement results in compression, or overriding, of stratum across the fault plane. |
| Review | Activity undertaken to determine the suitability, adequacy and effectiveness of the subject matter to achieve established objectives (ISO 31000 2009). |
| Rib | Side or sidewall of a coal pillar. |
| Ride | Down dip shear displacement of the roof of mine workings relative to the floor. |
| Rider | A thin seam of coal overlying a main coal seam. |
| Rill | The action of solid material flowing under gravity. |
| Risk | A combined measure of the consequences of an event and the likelihood that the event will occur. Risk may have positive or negative consequences. |
| | – |
| | The effect that uncertainty has on an organisation's objectives (ISO 31000 2009). |
| Risk Analysis | A process to comprehend the nature of risk and to determine the level of risk (ISO 31000 2009). |
| Risk Assessment | The overall process of risk identification, risk analysis and risk evaluation (ISO 31000 2009). |
| Risk Criteria | Terms of reference against which the significance of risk is evaluated (ISO 31000 2009). |
| Risk Management Process | The systematic application of management policies, procedures and practices to the activities of communicating, establishing the context, and identifying, analysing, evaluating, treating, monitoring and reviewing risk (ISO 31000 2009). |
| Risk Treatment | The process to modify risk (ISO 31000 2009). |
| Rock Mass | The sum total of the rock as it exists in situ. This includes intact rock material, groundwater, fractures, faults, dykes and other planes of weakness. |
| Rock Mechanics | Is the theoretical and applied science of the mechanical behaviour of rock and rock masses; it is that branch of mechanics concerned with the response of rock and rock masses to the force fields of their physical environment (Brady and Brown 2006, as offered by the US National Committee on Rock Mechanics). |
| Run-out | A long split driven from the main panel development to the flanks of a pillar extraction panel. |
| Scaling Down | The act of prising loose pieces of rock from the roof and ribsides using a bar. Also referred to as 'barring down'. |
| Scat | Small pieces of rock which fall from the surface of an excavation. Also referred to as 'brat'. |
| Seal | A substantial stopping or barrier constructed in a roadway to prevent or retain fluid flow. |
| Serviceability | Level of suitability, usefulness, and effectiveness in fulfilling required functions. |
| Sequence | The order in which pillars are developed and/or lifted off. |
| SFARP | So far as is reasonably practical |
| Shaft | A vertical or near vertical connection between the surface and the mining horizon. |

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| Shotcrete | A mortar or concrete mix sprayed onto a surface at high pressure using compressed air. |
| Shunt | (noun) – A temporary parking bay to permit one vehicle to pass another approaching from the opposite direction. |
| | – |
| | (verb) – The act of moving into a passing bay to permit another vehicle to pass. |
| Sill | A laterally extensive intrusion of igneous material. |
| Slabbing | See 'pillar stripping'. |
| Slickenside | A smooth, slippery sliding plane within the rock mass. |
| Slip | A well developed planar joint in a coal pillar. Slips are noted for their propensity to result in a rib fall. |
| Snook | A South African term for 'stook'. |
| Soft | In relation to strata, refers to materials that are more soil like and homogenous with little in the way of defects, so that the low uniaxial compressive strength (UCS) of these materials is due to the low strength of the intact material. |
| | – |
| | In relation to load and displacement, refers to a structure that has a low stiffness (such that a small increment in load results in a large increment in displacement). |
| Spalling | In rock mechanics and hard rock mining, the term describes stress-induced fracturing at the boundary of an excavation in regions of maximum tangential stress. |
| | – |
| | In coal mining, the term is used in a more general sense to encompass all unravelling and falling of material from the sides of a coal pillar. |
| Span | The shortest distance between two abutments. |
| Spile | (noun) – A bar or tube that is driven in over the top of weak or fallen ground. |
| | – |
| | (verb) – The process of driving a series of parallel spiles in close proximity to each other over the top of weak or fallen ground so as to form an artificial roof, or 'verandah,' and then incrementally providing support to this verandah as the material beneath it is removed. In the case of a fall of ground, the verandah is usually progressively supported by cross supports set on heavy legs. Where the material is yet to fall, tendon support may be utilised in place of, or as well as, cross supports and legs. |
| Split | A roadway developed within a pillar to divide it into smaller portions. Also referred to as a pocket in some situations. |
| Sprag | A support set against the face of a coal rib or sidewall to retain the face of the sidewall in place. Historically, sprags comprised short props set horizontally between the sidewall and vertical props. Most sprags now comprise some form of hydraulically activated steel face plate. |
| Squeeze | A controlled pillar system failure. Also referred to as a 'ride' or 'pillar run'. |
| Standard Operating Procedure | A documented way of working or an arrangement of facilities for the purpose of achieving an acceptable level of risk. |
| Strata Control | A term widely used in the mining industry before the development of the terms 'geomechanics', 'rock mechanics' and 'mining geomechanics'. It is still used in the coal mining industry to mean 'the control and prediction of strata behaviour |

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| | during development and extraction operations' (Brown 1998). |
| Stochastic | Process of determining likelihood on the basis of a random distribution of probabilities. |
| Stook | An term used in Australian pillar extraction operations to describe a remnant portion of a pillar. Referred to as a 'snook' in South Africa and as a 'stump' or a 'pushout' in the USA. |
| Stooping | Another term for pillar extraction. |
| Stopping | A wall or barricade built across a roadway to separate air courses in a mine. |
| Strike | The direction of a line that defines the intersection of a rock bed with a horizontal plane. |
| Strike-slip Fault | A fault plane in which the direction of movement is along the strike of the fault. |
| Stripping | See 'pillar stripping'. |
| Strong | In respect of rock, typically regarded as material with a uniaxial compressive strength >40–50 MPa. |
| Structure | See 'geological structure'. |
| Stump | A USA term for a stook. |
| Support | The application of a reactive force at the face of an excavation. Examples include timber props, shotcrete and backfill (adapted from Brown 1998). |
| Surge | The process of operating two shuttle cars in series, whereby one shuttle car transfers its load to another at some point along the wheeling route between the coal face and the panel conveyor belt. |
| Swilley | A localised depression in the working floor of a coal seam. Often associated with weak ground conditions. |
| Uni-directional cutting (Uni-di) | The process of cutting the longwall face to its full height in two passes of the shearer, with the first pass taking a cut from the upper portion of the coal face and the return pass cutting out the remainder of the face. |
| Tabular | Bedded and laterally extensive. |
| TARP | Trigger Action Response Plan. |
| Technology | An enabling package of knowledge, devices, systems, processes, and other technologies, created for a specific purpose (Brown 2001a). |
| Thixotropic | A material that exists in a gel state under static conditions but becomes fluid when shaken, stirred or otherwise stressed. |
| Threat | A means by which a hazard can materialise. |
| Thrust Fault | A reverse fault that dips at less than 45°. |
| Top Hat | A cross support utilized in coal mining that comprises a rolled steel channel of typically 4–10 mm wall thickness that has the cross-section of a top hat. The deep throat imparts a high moment of inertia relative to the weight of the beam. |
| Transversely Isotropic | Having the same physical properties and, therefore, the same reaction to stress in two orthogonal directions but not in the third direction. |
| Trigger | Risk Management – a predetermined type or magnitude of behaviour prompting intervention. |
| | – |
| | Physics – a threshold value which, if exceeded, results in instability that produces a sudden change in system properties. |
| Trigger Action Response Plan (TARP) | A plan designed to prevent a threat from escalating by identifying potential precursors, or triggers, to the threat event, assigning a hierarchy of alarms, or trigger levels, to each potential precursor, and specifying responses for each trigger level. |
| Unravelling | Progressive disintegration of the strata between ground support elements. Also referred to as 'ravelling'. |
| UNSW | University of New South Wales (Sydney, Australia). |
| Verandah | A false roof constructed to support overlying weak strata. |
| Water Crack | A wet or water bearing joint plane in the roof of coal mine workings. A water crack may dry up over time but still be detectable due to staining. |

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| Weak | In relation to strata, refers to materials that are not necessarily homogenous and have a low strength, typically in a uniaxial compressive strength range of 0.5–10 MPa, as a result of the very low strength of the intact material and/or because of a significant density of lower strength defects. |
| Web | A thin fender of coal left between two lifts, usually as a temporary support measure. Portions of a web may be extracted (pocketed) on retreat out of a lift. |
| Wheeling | A term used to describe the conveying of coal by means of mobile vehicles from the working face to the panel conveyor belt. |
| Wheeling Road | Roadways used by mobile vehicles for conveying coal from the coal face to the panel conveyor belt. |
| Winded Coal | Coal that has been destressed, usually as a result of failing. |
| Windrow | A pile of loose rock placed as a barrier down the sides or middle of a surface roadway. |
| | – |
| | Loose rock material that spills from the blade or shovel of mining equipment such as bulldozers, graders and continuous miners or which is pushed up in front of the blade or shovel. |

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# Glossary of Symbols

| Symbol | Description | Dimensions |
|---|---|---|
| a | An input parameter to the Hoek-Brown failure criterion | Dimensionless |
| A | Area over which a force acts | $m^2$ |
| | Cross-sectional area | |
| $A_c$ | Tributary area of a pillar (in plan view) | $m^2$ |
| $A_i$ | Area beneath a stress or pressure profile (which equates to total force) | N |
| $A_m$ | Area of workings extracted (exposed roof area) | $m^2$ |
| $A_p$ | Cross-sectional area of a pillar (in plan view) | $m^2$ |
| $A_1$, $A_2$, | Area beneath a stress or pressure profile (which equates to total force) | N |
| b | Beam width | m |
| | Bord or roadway width | m |
| | Aperture of a rock fracture | m |
| $b_1$ | Bord or roadway width at right angles to longest pillar side | m |
| $b_2$ | Bord or roadway width at right angles to shortest pillar side | m |
| B | Footing diameter or width | m |
| $B_i$ | Area beneath a stress or pressure profile (which equates to total force) | N |
| c | Cohesion | $N/m^2$ |
| | Distance from central axis about which bending occurs in a column or beam | m |
| $c_r$ | Residual cohesion | $N/m^2$ |
| $c_1$ | Cohesion of upper layer of soft floor material | $N/m^2$ |
| $c_2$ | Cohesion of lower layer of soft floor material | $N/m^2$ |
| C | Compression | Dimensionless |
| $C_\alpha$ | Secondary compression index | Dimensionless |
| $C_c$ | Compressibility index | Dimensionless |
| $C_p$ | Circumference of a pillar | m |
| $C_v$ | Coefficient of consolidation | $m^2/s$ |
| $C_1$ | Pillar centre distance parallel to shortest pillar side | m |
| | $w_1 + b_1/\sin \theta = w_1 + b_1 \mathrm{cosec}\, \theta$ | |
| $C_2$ | Pillar centre distance parallel to longest pillar side | m |
| | $w_2 + b_2/\sin \theta = w_2 + b_2 \mathrm{cosec}\, \theta$ | |
| d | Distance | m |
| | Specimen diameter | m |
| | Depth of a footing beneath the surface | m |
| $d_T$ | Diameter of a tendon | m |
| D | Lateral extent of side abutment zone | m |
| $D_d$ | Depth to base of a dolerite sill | m |
| e | Areal extraction, expressed as a fraction | m |
| | Distance from neutral axis to point of (eccentric) loading on a column | m |
| | Pre-existing eccentricity in a column | Dimensionless |

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| Symbol | Description | Dimensions |
|---|---|---|
| $e_o$ | Void ratio | Dimensionless |
| $E$ | Modulus | $N/m^2$ |
| $E_c$ | Elastic modulus of coal | $N/m^2$ |
| $E_{eq\text{-}n}$ | Equivalent elastic modulus normal to bedding | $N/m^2$ |
| $E_{eq\text{-}p}$ | Equivalent elastic modulus parallel to bedding | $N/m^2$ |
| $E_f$ | Modulus of deformation of backfill or caved material | $N/m^2$ |
| $E_o$ | Effective elastic modulus of the overburden | $N/m^2$ |
| $E_s$ | Secant modulus | $N/m^2$ |
| $E_t$ | Tangent modulus | $N/m^2$ |
| $E_T$ | Elastic modulus of tendon material | $N/m^2$ |
| $E_{ti}$ | Initial tangential modulus of backfill material | $N/m^2$ |
| $E_u$ | Undrained modulus | $N/m^2$ |
| $f_1$ | Moment arm distance to force $F_1$ | m |
| $f_2$ | Moment arm distance to force $F_2$ | m |
| $F$ | Force (driving or reaction) | N |
| $F_{pt}$ | Pretension force applied to a tendon | N |
| $F_H$ | Horizontal force | N |
| $F_1$, $F_2$ | Driving forces | N |
| $g$ | Gravitational acceleration constant | $m/s^2$ |
| $G_e$ | Shear modulus of encapsulating grout | $N/m^2$ |
| GSI | Geological Strength Index | Dimensionless |
| $h$ | Specimen height | m |
| | Mining height or pillar height | m |
| $h_c$ | Height of caving | m |
| $h_o$ | Height of a reference body used in power pillar strength formulae | m |
| $H$ | Depth of mining (measured to either top of seam or bottom of seam) | m |
| | Length of drainage path | m |
| $H_d$ | Height of complete groundwater drainage | m |

| Symbol | Description | Dimensions |
|---|---|---|
| $H_f$ | Head of fluid acting at roof level or height of fluid above roof level | m |
| $H_{\text{solid rock}}$ | Solid rock head cover to working horizon | m |
| $i$ | A friction angle that accounts for asperities on a fracture surface, or angle of saw tooth asperity faces | Degrees |
| $I$ | Second moment of inertia | $m^4$ |
| $I_p$ | A footing influence factor | Dimensionless |
| $J_a$ | Joint alteration number | Dimensionless |
| $J_n$ | Joint set number | Dimensionless |
| $J_r$ | Joint roughness number | Dimensionless |
| $J_w$ | Joint water reduction factor | Dimensionless |
| JRC | Joint roughness coefficient | Dimensionless |
| JCS | Joint wall compressive strength | $N/m^2$ |
| $k$ | Stiffness | N/m |
| | Horizontal to vertical stress ratio | Dimensionless |
| | Bulking factor of caved strata | Dimensionless |
| | An adjustment factor to account for dimensions of input parameters | Dimensionless |
| | Permeability | $m^2$ |
| $k_{bond}$ | Stiffness of encapsulating medium | N/m |
| $k_f$ | Stiffness of backfill or caved material | N/m |
| $k_i$ | Initial bulking factor | Dimensionless |
| $k_t$ | Tendon stiffness | N/m |
| $k_1$ | Compressive strength of a reference cube of coal used in linear pillar strength formulae | $N/m^2$ |
| $k_2$ | Strength of a reference body of coal used in power pillar strength formulae | $N/m^2$ |
| $K$ | Strength magnification factor due to confinement | Dimensionless |
| | Effective length coefficient for a column | Dimensionless |
| | Hydraulic conductivity or coefficient of permeability | m/s |

(continued)

| Symbol | Description | Dimensions |
|---|---|---|
| $K_1$, $K_2$, $K_3$, $K_4$ | Proportionality factors associated with classical beam theory formulations for various beam and column loading configurations and end constraints | Dimensionless |
| l | Length of a fracture orthogonal to flow | m |
| $l_{LT}$ | Load transfer distance along a tendon | m |
| L | Length of a tendon | m |
|  | Length of a footing | m |
| $L_e$ | Effective length of a column or beam | m |
| $L_p$ | Pillar load | N |
| $L_{P\ peak}$ | Peak load capacity of a tendon plate assembly | N |
| $L_s$ | Total side abutment load based on abutment angle concept | N |
| $L_T$ | Tensile force generated in a tendon | N |
| $L_{T\ peak}$ | Ultimate tensile capacity of a tendon | N |
| $L_{T\ yield}$ | Yield load of a tendon | N |
| $L_{up}$ | Load acting upwards on the roof of mine workings | N |
| $m_b$ | An input parameter to the Hoek-Brown failure criterion | Dimensionless |
| $m_\varepsilon$ | Strain magnification factor | Dimensionless |
| $m_v$ | Coefficient of volume compressibility | $m^2/N$ |
| M | Moment couple or internal bending moment | Nm |
| $M_{ext}$ | External bending moment | Nm |
| $M_{int}$ | Internal bending moment | Nm |
| n | Number of data points | Dimensionless |
|  | Number of tendons per square metre | $Tendons/m^2$ |
| N | Number of tendons per row | Dimensionless |
| $N_c$ | A bearing capacity factor | Dimensionless |

| Symbol | Description | Dimensions |
|---|---|---|
| $N_q$ | A bearing capacity factor | Dimensionless |
| $N_\gamma$ | A bearing capacity factor | Dimensionless |
| p | Hydrostatic stress | $N/m^2$ |
|  | Applied overburden stress | $N/m^2$ |
| $p_c$ | A material constant | $N/m^2$ |
| P | Load applied axially to a column | N |
|  | Ventilation fan pressure | $N/m^2$ |
| $P_{cr}$ | Critical axial load or Euler critical load | N |
| $P_{Ra}$ | Average roof contact pressure of a powered support canopy | $N/m^2$ |
| $P_{Fa}$ | Average floor contact pressure of a powered support base | $N/m^2$ |
| q | Uniformly distributed load on a structure (beam, footing etc.) | $N/m^2$ |
| $q_u$ | Bearing capacity of a foundation | $N/m^2$ |
| Q | Fluid flow rate (ventilation air, groundwater etc.) | $m^3/s$ |
| r | Correlation coefficient | Dimensionless |
|  | A dimensionless constant used in linear pillar strength formulae | Dimensionless |
|  | Least radius of gyration for a column | m |
| $r_1$ | Moment arm distance to reaction force $R_1$ | m |
| $r_2$ | Moment arm distance to reaction force $R_2$ | m |
| $r_2$ | Coefficient of determination | Dimensionless |
| R | Pillar width-to-height ratio | Dimensionless |
|  | Spacing between rows of tendons | m |
|  | Roadway (ventilation) resistance | $Ns^2/m^8$ |
| $R_{min}$ | Minimum pillar width-to-height ratio (wmin/h) | Dimensionless |
| $R_o$ | The width-to-height ratio at which a pillar is considered to be ‘squat’ | Dimensionless |

(continued)

| Symbol | Description | Dimensions |
|---|---|---|
| $R_l$ | The width-to-height ratio at which rectangular pillar strength benefits first start to materialise | Dimensionless |
| $R_u$ | The width-to-height ratio at which rectangular pillar strength benefits are maximised | Dimensionless |
| $R_1$ | Radius of curvature | m |
|  | Resultant force | N |
| $R_2$ | Radius of curvature | m |
|  | Resultant force | N |
| s | An input parameter to the Hoek-Brown failure criterion | Dimensionless |
|  | Standard deviation | Dimensionless |
| $s^2$ | Variance or second moment | Dimensionless |
| S | Span of a beam | m |
|  | Span of an excavation | m |
|  | A fictitious balancing or stabilising force | N |
| $S_c$ | A foundation engineering shape factor | Dimensionless |
| $S_e$ | Elastic settlement | m |
| $S_p$ | Primary consolidation settlement | m |
| $S_q$ | A foundation engineering shape factor | Dimensionless |
| $S_s$ | Secondary consolidation settlement | m |
| $S_t$ | Ultimate tensile strength of a tendon | $N/m^2$ |
| $S_\gamma$ | A foundation engineering shape factor | Dimensionless |
| t | Thickness of a beam, column or plate | m |
|  | Thickness of foundation layer | m |
|  | Width of tendon/width annulus | m |
| $t_d$ | Thickness of a bridging superincumbent strata | m |
| $t_p$ | Thickness of parting between mining horizon and base of a dolerite sill | m |
| $t_1$ | Thickness of upper layer of soft floor material | m |
| $t_2$ | Thickness of lower layer of soft floor material | m |
| $t_{100}$ | Foundation layer thickness at end of primary consolidation | m |
| $T_x$ | Time to reach x% consolidation | s |
| $T_{F(x)}$ | Time factor associated with x% consolidation | Dimensionless |
| T | Tension | Dimensionless |
| $T_f$ | A tensile strength reduction factor, typically in the range of 10–30 | Dimensionless |
| $T_1$ | Time when secondary consolidation is assumed to begin | s |
| $T_2$ | Time for which secondary settlements are calculated | s |
| TSF | Tectonic Stress Factor or tectonic induced strain | Dimensionless |
| u | Pore pressure | $N/m^2$ |
|  | Hydrostatic pressure | $N/m^2$ |
| U | Potential energy | J |
| $U_{hi}$ | Inward deflection of the abutments of a beam under the effect of a lateral load. | m |
| $U_{vi}$ | Deflection at the mid-span of a beam or column due to the abutments deflecting inwards by an amount $U_{hi}$ | m |
| $U_{xi}$ | Deflection of a beam or column at any point x along its axis | m |
| V | Material volume | $m^3$ |
|  | Pillar volume | $m^3$ |
|  | Transverse shear force | N |
| $V_x$ | Transverse horizontal surface displacement | m |
| $V_y$ | Longitudinal horizontal surface displacement | m |
| $V_z$ | Vertical surface displacement | m |
| $V_{z\ max}$ | Maximum vertical surface displacement | m |
| w | Specimen width | m |
|  | Pillar width | m |
|  | Width of a footing | m |

(continued)

| Symbol | Description | Dimensions |
|---|---|---|
| $w_{eff}$ | Effective width of a parallelepiped shaped pillar when equated to the strength of a square pillar | m |
| $w_{min}$ | Minimum pillar width (normal to long side of pillar) $= w_1 \text{ Sin } \theta$ | m |
| $w_o$ | Width of a reference body used in power pillar strength formulae | m |
| $w_1$ | Length of shortest side of a pillar | m |
|  | Length of shortest side of a foundation | m |
| $w_2$ | Length of longest side of a pillar | m |
|  | Length of longest side of a foundation | m |
| W | Width of an excavation – rib to rib | m |
|  | Maximum weight of a detached block that a longwall powered support can sustain without going into yield | N |
| $W_c$ | Critical panel width | m |
| $W_e$ | Strain energy per unit volume | $J/m^3$ |
| $W_o$ | The overall extent (width) of a series of adjacent mining panels | m |
| $W_p$ | Overall width of a panel of pillars | m |
| $W_1$ | Stored elastic strain energy resulting from an application of a tendon force, F | $J/m^3$ |
| $W_2$ | Stored elastic strain energy resulting from an application of a tendon force, $F_x$ | $J/m^3$ |
| x | An independent variable | Dimensionless |
| $\bar{x}$ | Mean x value, first moment or centre of gravity | Dimensionless |
| $x_i$ | The $i^{th}$ data value | Dimensionless |
| y | A dependent variable | Dimensionless |
| $\bar{y}$ | Mean y value | Dimensionless |
| z | The normal distance from the neutral axis to a point, p. | m |
| $z'$ | Deviation in a column | m |
| $\alpha$ | Angle of draw, measured from the vertical | Degrees |
|  | Angle of break, measured from the vertical, of a detached roof block on a longwall face | Degrees |
|  | The power to which the width parameter is raised in power pillar strength formulae | Dimensionless |
| $\alpha_u$ | A pore pressure reduction factor | Dimensionless |
| $\beta$ | Angle of break or caving angle over the goaf measured from the horizontal (or mining horizon) | Degrees |
|  | The power to which the height parameter is raised in power pillar strength formulae | Dimensionless |
| $\gamma$ | Specific weight, or unit weight | $N/m^3$ |
|  | Shear strain or horizontal distortion | Dimensionless |
| $\gamma_w$ | Unit weight of water | $N/m^3$ |
| $\delta$ | Deflection | m |
| $\delta_{b'}$ | Deflection of a beam when supported at its mid-span | m |
| $\delta_b$ | Deflection of a beam that is not supported at its mid-span | m |
| $\delta_{max}$ | Maximum deflection | m |
| $\varepsilon$ | Strain | Dimensionless |
|  | A measure of the rate of squat pillar strength increase once wm/h exceeds $R_o$ | Dimensionless |
| $\varepsilon_a$ | Axial strain | Dimensionless |
| $\varepsilon_c$ | Compressive strain | Dimensionless |
| $\varepsilon_f$ | Critical failure strain | Dimensionless |
| $\varepsilon_H$ | Lateral pillar strain | Dimensionless |
| $\varepsilon_l$ | Lateral strain | Dimensionless |
| $\varepsilon_{lp}$ | Lateral pillar strain | Dimensionless |
| $\varepsilon_m$ | Maximum strain that can be developed in backfill | Dimensionless |
| $\varepsilon_t$ | Tensile strain | Dimensionless |

(continued)

| Symbol | Description | Dimensions |
|---|---|---|
| $\varepsilon_V$ | Vertical pillar strain – ashfill research | Dimensionless |
| $\theta$ | Abutment angle, measured from the vertical | Degrees |
| | The smaller internal angle of a parallelepiped shaped pillar ($\theta \leq 90°$) | Degrees |
| $\lambda$ | Slenderness ratio of a column | Dimensionless |
| $\mu$ | Coefficient of friction = Tan($\phi$) | Dimensionless |
| $\mu_d$ | Dynamic viscosity | kg/ms |
| $\mu_R$ | Resin/rock coefficient of friction | Dimensionless |
| $\mu_T$ | Tendon/resin coefficient of friction | Dimensionless |
| $\nu$ | Poisson's ratio | Dimensionless |
| $\rho$ | Density | $kg/m^3$ |
| $\rho_o$ | Effective (overall) density of the overburden | $kg/m^3$ |
| $\rho_f$ | Density of fluid | $kg/m^3$ |
| $\sigma$ | Stress (including overburden stress) | $N/m^2$ |
| $\sigma'$ | Effective stress | $N/m^2$ |
| $\sigma_a$ | Axial stress | $N/m^2$ |
| $\sigma_{aps}$ | Average pillar stress | $N/m^2$ |
| $\sigma_{ax}$ | Abutment stress at distance of x metres from the edge of an excavation | $N/m^2$ |
| $\sigma_A$ | Lateral ashfill pressure | $N/m^2$ |
| $\sigma_c$ | Uniaxial compressive stress | $N/m^2$ |
| $\sigma_{c50}$ | Uniaxial compressive stress of a 50 mm diameter specimen | $N/m^2$ |
| $\sigma_{ci}$ | Intact compressive strength | $N/m^2$ |
| $\sigma_f$ | Fibre stress in a beam, column or plate | $N/m^2$ |
| | Backfill reaction stress | $N/m^2$ |
| | Confining pressure developed by backfill or caved material | $N/m^2$ |
| $\sigma_h$ | Horizontal stress | $N/m^2$ |
| $\sigma_{hp}$ | Horizontal primitive stress | $N/m^2$ |
| $\sigma_{ips}$ | Induced average pillar stress | $N/m^2$ |
| $\sigma_l$ | Lateral stress | $N/m^2$ |
| $\sigma_{max\ comp}$ | Maximum compressive fibre stress in a beam, column or plate | $N/m^2$ |
| $\sigma_{max\ ten}$ | Maximum tensile fibre stress in a beam, column or plate | $N/m^2$ |
| $\sigma_n$ | Normal stress | $N/m^2$ |
| $\sigma'_n$ | Effective normal stress | $N/m^2$ |
| $\sigma_0$ | Axial stress in a tendon | $N/m^2$ |
| $\sigma_p$ | Vertical pillar stress – ashfill research | $N/m^2$ |
| $\sigma_{ps}$ | Pillar strength | $N/m^2$ |
| $\sigma_{rr}$ | Radial stress | $N/m^2$ |
| $\sigma_v$ | Vertical stress | $N/m^2$ |
| $\sigma_{ve}$ | Vertical effective stress | $N/m^2$ |
| $\sigma_{vp}$ | Vertical primitive stress | $N/m^2$ |
| $\sigma_y$ | Yield stress | $N/m^2$ |
| $\sigma_1$ | Maximum, or major, principal stress | $N/m^2$ |
| | Triaxial strength | $N/m^2$ |
| $\sigma_2$ | Intermediate principal stress | $N/m^2$ |
| $\sigma_3$ | Minimum, or minor, principal stress | $N/m^2$ |
| | Confining stress | $N/m^2$ |
| $\sigma'_1$ | Maximum principal effective stress at failure | $N/m^2$ |
| $\sigma'_3$ | Minimum principal effective stress at failure | $N/m^2$ |
| $\sigma_{\theta\theta}$ | Tangential stress | $N/m^2$ |
| $\tau$ | Shear stress | $N/m^2$ |
| $\tau_{max}$ | Maximum shear stress | $N/m^2$ |
| $\tau(x)$ | Shear stress at a distance x along a tendon | $N/m^2$ |
| $\tau_c$ | The cohesive component of shear resistance | $N/m^2$ |
| $\tau_f$ | The frictional component of shear resistance | $N/m^2$ |
| $\tau_r$ | Residual shear strength | $N/m^2$ |
| $\tau_T$ | Total shear resistance | $N/m^2$ |
| $\tau_{xz\ ave}$ | Average shear stress acting in z direction of plane yz | $N/m^2$ |
| $\phi$ | Angle of friction | Degrees |
| | Effective abutment angle over the goaf, measured from the vertical | Degrees |

(continued)

| Symbol | Description | Dimensions |
|---|---|---|
| $\phi_b$ | Base friction angle or angle of friction on a smooth surface | Degrees |
| $\phi_d$ | Dynamic angle of friction | Degrees |
| $\phi_r$ | Residual friction angle | Degrees |
| $\phi_s$ | Static angle of friction | Degrees |
| $\varphi$ | Angle through which the end of a beam or column may rotate | Degrees |
| $\Delta d$ | Displacement | m |
| $\Delta h$ | Change in pillar height (pillar compression) or change in mining height | m |
|  | Head difference along a flow path | m |
| $\Delta l$ | Distance/dilation across a parting place | m |
|  | Distance between measurements of head | m |
| $\Delta P$ | Incremental axial force applied to a column | N |
| $\Delta\sigma_{vi}$ | Induced vertical stress | $N/m^2$ |
| $\Theta$ | $= \Theta_O^{\frac{R-R_l}{R_u-R_l}}$ | Dimensionless |
| $\Theta_o$ | $=2w_2/(w_1 + w_2)$ | Dimensionless |

## Symbols in Metric System

|  |  |  |  |
|---|---|---|---|
| nano | n | $10^{-9}$ | billionth |
| micro | μ | $10^{-6}$ | millionth |
| milli | m | $10^{-3}$ | thousandth |
|  |  | $10^{0}$ | (=1 unit) |
| kilo | k | $10^{3}$ | thousand |
| mega | M | $10^{6}$ | million |
| giga | G | $10^{9}$ | billion |

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